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CONSTRUCTION WORK ON THE CANADIAN NORTHERN RAILWAY SYSTEM.

In the active railway construction in Canada during the past few years the Canadian Northern Ry. has been very prominent, and during the past season it has completed its new and important main line through what was formerly the Northwest Territory, but which is now divided up into the provinces of Saskatchewan and Alberta. This line opens up a new region to settlement and gives a new outlet for grain from the Northwest to lake navigation at Fort William.

The Canadian Northern Ry. has developed from very small beginnings until it has become one of the principal railways of Canada, which has already an aggregate length of about 2,500 miles (exclusive of its lines in the maritime provinces), and the prominent feature of which is the development of the Northwest region far to the north of the main line of the Canadian Pacific Ry. This region is being settled rapidly by immigrants from Europe and also from the United States. A map of the system is shown in Fig. 1. From Lake Superior at Fort William, Ont. (where it has extensive terminals and a concrete grain elevator of 7,000,000 bushels capacity), the line extends to Winnipeg (439 miles), the headquarters of the system. From Winnipeg the line extends to Dauphin (178 miles), whence two practically parallel lines are run. One of these continues to the northwest corner of Manitoba and thence almost due west to Prince Albert (540 miles from Winnipeg), which was reached by the track in January, 1906. The other line runs northwest from Dauphin to Edmonton (825 miles from Winnipeg), and this point was reached by the track in November, 1905. The company has a charter for a line from Edmonton to the Peace River, the surveyed route for which gives a low-level pass through the Rocky Mountains. The continuation of the line to the Pacific coast seems most probable, as stated in an article on Canadian transcontinental railways in our issue of Oct. 22, 1903, but at the present time the rapid settlement of the country along the existing line makes this coast extension of rather secondary importance.

The company owns the line of the old Port Arthur, Duluth & Western Ry., which was originally intended to connect Port Arthur with Duluth, but was never completed. From Winnipeg it has a line south to the international boundary at Emerson, where connection is made with both the Great Northern Ry. and the Northern Pacific Ry. for through trains between St. Paul and Winnipeg. In the southern and comparatively settled portion of Manitoba there are several lines, which are being extended from time to time. There is also the old Hudson's Bay Ry. from Winnipeg to Lake Manitoba, and it is expected that active work will soon be commenced on a line from the Prince Albert branch to Fort Churchill, on Hudson's Bay. Besides its lines in the Northwest, the company has important Eastern lines. It owns the Great Northern Ry. of Canada, from Hawkesbury, on the Ottawa River, to Quebec, with a branch into Montreal, 250 miles in all. It has a charter for a line from Hawkesbury west to Ottawa and Parry Sound (300 miles), and is building a line from Toronto north to Sudbury (240 miles), where it will connect with the Canadian Pacific Ry. This line is the beginning of a proposed route to James Bay (or Hudson's Bay). In addition, it has a charter for a railway from Sudbury to Fort William, to connect the eastern and northwestern portions of the system; but, as this would be through a wild and unproductive region which does not afford traffic for even one railway, it is thought probable that arrangements may be made for running powers over the Canadian Pacific Ry. between these points. With these connections completed between Port Arthur and Hawkesbury, the company will have a through route from the Northwest, not only to the Great Lakes, but also to the St. Lawrence River ports of Montreal and Quebec. The company also owns some small lines in Nova Scotia, which may eventually form part of a route from the St. Lawrence to the summer ports of Halifax and St. John.

Regardless of future developments, however, the existing line through the Northwest and Manitoba and terminating on Lake Superior forms in itself an important trunk line, opening up practically new territories and affording additional facilities for the handling of the immense and increasing grain traffic. We are not immediately concerned with the agricultural side of the matter, but it is worthy of note that a region which 20 years ago was thought to be far beyond the line of cultivation is now found to be capable of growing enormous quantities of wheat of the finest quality, and the settlements are being pushed farther and farther northwards every year.

FIG. 3. TRACKLAYING TRAIN ON TEMPORARY PILE BRIDGE OVER THE SASKATCHEWAN RIVER AT FORT SASKATCHEWAN, CANADIAN NORTHERN RY.

The development of this railway system has been mainly the work of Mr. William Mackenzie, of Toronto, who is the President of the railway company. He is also President of the Mackenzie & Mann Construction Co., which has the general contract for the construction of the new lines. Mr. R. J. Mackenzie is Managing Director of Western Lines; Mr. M. H. McLeod, Chief Engineer, and Mr. H. Bannister and Mr. W. L. Mackenzie, Bridge Engineers. To the officers we are indebted for much information, and for courtesies shown during a recent trip to the head of the new line.

SURVEYS.

The methods and force employed in reconnoissance depend largely on the nature of the country, whether the work is in the bush or on the prairie. In the latter case, there is usually an engineer with one assistant, a saddle horse and a team with driver. The location is usually made in the field, the party consisting of the following: Engineer-in-charge, transitman, leveler, topographer, rodman, two chainmen, head picketman and three or four axemen, cook and cook's assistant, and usually two teams and a saddle horse. The final location is made after an inspection on the ground by the chief engineer or the principal assistant engineer. The location was usually made to some fixed point in advance of any construction, and the grading was kept about fifty miles in advance of the tracklaying. The telegraph line was usually kept completed to within from 5 to 10 miles behind the tracklaying gang.

CONSTRUCTION WORK.

That part of the main line of the Canadian Northern Ry. which has been built during the past season is the stretch from Humbolt to Edmonton. Work was commenced west from Humbolt in March, 1905, and was completed as far as Edmonton (400 miles) in November; since then it has been continued westward as long as weather permitted. The grading is not heavy, averaging about 15,000 cu. yds. per mile; but the line cuts across the general direction of the streams and watercourses, and as these are generally in deep ravines there are a number of high banks and moderate-sized trestles. The country is partly flat, but mainly rolling prairie, making a continuous series of short cuts and fills between the ravines. There are only four large steel bridges, but these are all notable structures, and will be referred to more fully later on. All of these cross the Saskatchewan River, and it may be noted that there is navigation from Edmonton by this river and Lake Winnipeg to the city of Winnipeg, about 1,000 miles. As the railway runs through an entirely new country, the towns are few and small, but they are rapidly increasing, both in number and population.

The maximum grades between Winnipeg and Edmonton are 0.5% eastbound and 1% westbound, with a maximum curvature of 6°. The grades are long and continuous on much of the line west of Humbolt, as the country has a general upward slope to the west. Between Winnipeg and Fort William the maximum grades are also as above noted, but with maximum curves of 8½°. The following list of elevations above sea level will give an idea of the general conditions:

	Distance, Miles.	Elev. ft.		Distance Miles.	Elev. ft.
Fort William	...	612	Humbolt	864	1,822
Winnipeg	439	751	Battleford	1,011	1,654
Dauphin	617	957	Vermilion	1,124	1,975
Kamsack	718	1,414	Edmonton	1,200	2,125

The grading was mainly done by drag scrapers, steam shovels being only occasionally used. The earth is a soft black loam, underlaid by clay or gravel (with many stones and small boulders in this underlying material); in some places, however, the ground is of loose sand. The culverts are of squared timbers for the sides, with a roof of cross logs notched to fit partly between the timbers and so hold them apart. Cross logs are also put beneath the timbers at the ends of the culvert and at intermediate points. Both pile and frame trestles are used, with the same floor construction as the bridges, described further on. The width of roadbed is 16 ft. on fills and 22 ft. in cuts. The ties are of hewed tamarack, not barked, and are obtained along the road (although there is very little large timber). There are 17 ties to a 30-ft. rail. The rails are of the Am. Soc. C. E. section, 60 lbs. per yd., and are curved by rail benders for curves of 3° and over. They are laid with square suspended joints (broken joints on curves), and spliced with 24-in. four-bolt angle bars. Split switches with 15-ft. switch rails and rigid switchstands are used. The passing sidings are about six to eight miles apart, 2,000 ft. long, 15 ft. c. to c. from main track, and having No. 9 frogs. At each siding there is also an elevator track, 45 to 60 ft. from the main track. Along the older part of the line, woven wire fencing has been erected, on posts 30 ft. apart, and the track is ballasted with an inferior kind of gravel, having fine sand and many large stones. On this part of the line, also, surface cattleguards with longitudinal steel slats are being put in, with wing fences and triangular apron fences. On the newer parts of the line, however, the ballast consists of the earth, sand or gravel of the roadbed. The telegraph line (which followed about 5 or 10 miles behind the grading) has poles spaced 176 ft. apart, or 30 to the mile,

The water stations are about 20 miles apart, but until these are completed on the new lines there is a tank car to each train, a rectangular wooden tank being mounted on a flat car coupled to the tender. The water is taken from wells and streams and from the lakes which abound along parts of the line, but the water is generally very hard. The water stations have wooden tanks of 40,000 gallons capacity, except that at division points (about 125 miles apart) 60,000-gallon tanks are used. The tanks are on timber towers, and the towers and tanks are enclosed by sheathing to exclude snow and to form a protection against freezing, as the temperature falls as low as 45° below zero and the winters are long. The general construction resembles that of the Canadian Pacific Ry. tanks and towers, illustrated in our issue of Nov. 2, 1905. The pipes are as usual enclosed in frost-proof boxing. At the back of the tower is the pump house, and gasoline engines of 5 HP. with geared pumps of 6,000 gallone capacity per hour are mainly used, owing to the expense of fuel for boilers of steam pumps. To prevent the water from freezing, there is a stove underneath the tank, with stove pipe passing through the roof of the tank.

The stations, of course, are small and of frame construction, and usually have living quarters for the station agent and telegraph operator. At Winnipeg, a handsome union station is to be built, and negotiations are under way for its use by the Canadian Northern Ry., Grand Trunk Pacific Ry., Great Northern Ry. and Northern Pacific Ry. The architect of the building is Mr. W. Warren, of New York, but contracts for the construction have not yet been awarded. The estimated cost is about $2,000,000, while the passenger and freight yards and other terminal facilities will bring the total cost of the Winnipeg terminal to about $3,000,000.

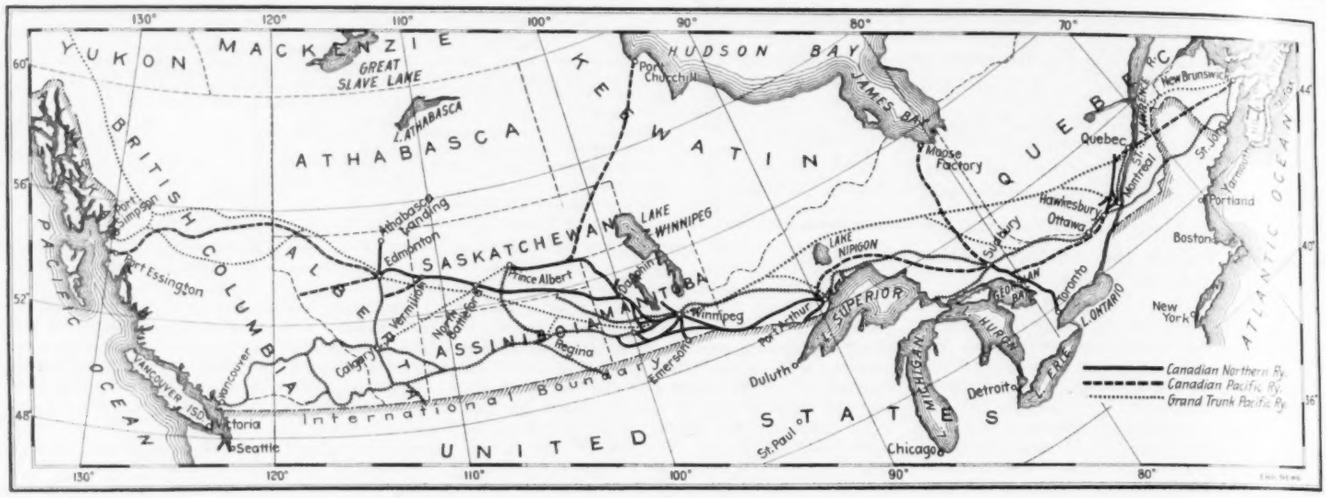


FIG. 1. MAP OF THE CANADIAN NORTHERN RAILWAY SYSTEM.

TRACKLAYING.

The tracklaying west of Humbolt was commenced on March 29, 1905, and the track was laid into Edmonton, 400 miles, on Nov. 24. The tracklaying train, Fig 2, consisted of the pioneer car at the head, then the flat cars of rails, and then the engine, behind which were the cars of ties and bridge material. The full number of ties are laid ahead of the train, and the rails are spiked at joints, quarters and centers. The work of completing the bolting and spiking was kept close to the rear of the tracklaying train. On the pioneer car was an elevated platform or cabin for a man with a flag to signal the engineman

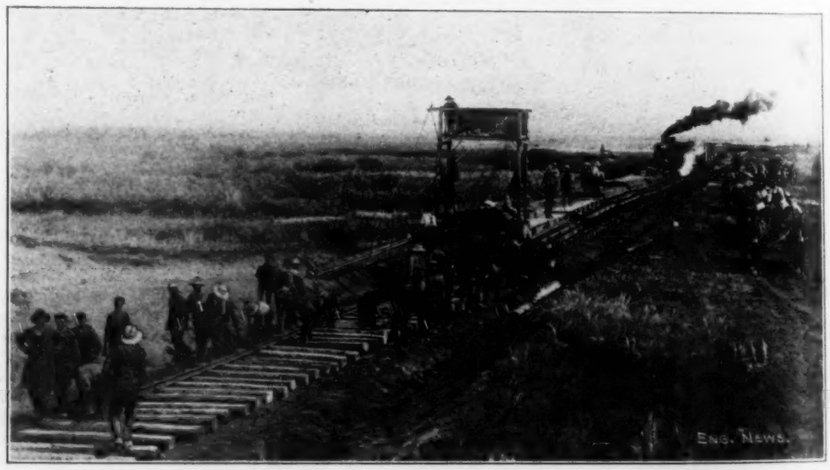


FIG. 2. TRACKLAYING TRAIN ON THE CANADIAN NORTHERN RY.

when to move ahead as each rail length of track was laid. The rails were run to the front in roller-ways or chutes attached to the sides of the flat cars. These chutes were about 30 ft. long, composed of two timbers 3 × 8 ins., 16 ins. apart, held together by bolts with spacing sleeves; journaled in the side timbers were 4-in. rollers (having collars on the ends), about 3 ft. apart. The ties were hauled ahead of the train by teams, about 50 teams being employed; these carried loads of 25 ties on the average, but the number varied, depending upon the nature of the ground over which they had to be hauled. The sidings for material trains were about 7 miles apart. The boarding train consisted of sleeping and dining cars, timekeeper's car with stores, supply cars, and a car for the general foreman. Double-deck sleeping cars were used on some of these trains.

All bridge material, such as piles, posts, caps, stringers, etc., was kept on the rear of the track-laying train and hauled ahead by teams to the various structures. The timber was erected by bridge gangs ahead of the track, and the track-laying train crossed the bridges when laid with half the complement of stringers and common track ties. The additional stringers, the proper bridge ties (12 ft. long), and the guard rails, were placed by a gang in the rear of the tracklaying gang.

The tracklaying gang was exceptionally large, averaging about 240 men, and worked from 7 a. m. to 6 p. m. It laid an average of 2.25 miles per day, or 17 miles per week, the maximum record being 3.7 miles in one day. Three (and sometimes four) gangs followed the tracklayers, surfacing and lining the track; each of these gangs consisted of 80 to 100 men.

The distribution of men in the tracklaying gangs for laying 3½ miles of track per day is given below; these aggregate 208, and the additional men kept on hand to lay sidings and make repairs to track behind the tracklaying gang proper would bring the full force up to about 240 men, the number given above:

TRACKLAYING GANG: CANADIAN NORTHERN RY.

In front of engine		In rear of engine	
1	Foreman.	1	Foreman.
8	Rail pullers.	24	Spikers.
2	Line men.	12	Bolters.
14	Tie men.	6	Nippers.
4	Spikers.	3	Spike distributors.
2	Nippers.	1	Putting in bolts
18	Steel men (handling rails on both sides of car).	1	Water boy.
		6	Liners.
		6	Tie men.
1	Liner.	80	Men handling ties, including teamsters.
1	Gage man.	1	Blacksmith.
1	Tie marker.	2	Cooks.
4	Bolters.	4	Helpers.
4	Loading steel on trams.	1	Time and storekeeper.
60	Total.	148	Total.

SASKATCHEWAN RIVER BRIDGES.

As already stated, the only important steel structures are the four large bridges over the Saskatchewan River, which flows through a deep valley, largely in sandy soil. The bottom is of sand, affording good hold for pile foundations. All the piers and abutments are of monolithic concrete construction. The spans are mainly deck trusses, and all the bridges have long and high trestle approaches. The bridge and trestle floors have ties of sawed pine, 8 × 8 ins., 12 ft. long, spaced 12 ins. c. to c. There are two guard rails on each side, both boxed out ½-in. over the ties. The one next the rail (10 ins. clear from the gage side of the rail head) is 5 × 8 ins., laid flat, and spiked to the ties; the outer one (36 ins. from the gage side of the rail) is 8 × 8 ins. and bolted to the ties.

The specifications require the use of medium open-hearth steel, having an ultimate strength of 60,000 to 68,000 lbs. per sq. in., and an elastic limit of at least 32,000 lbs. per sq. in.

1. BRIDGE AT CLARK'S CROSSING. (Crossing of the South Branch, 485 miles west of Winnipeg.)—This bridge is designed for a live load of two consolidation engines of 306,000 lbs. followed by a train of 4,200 lbs. per lin. ft. The dead load is 2,400 lbs. per lin. ft. There are six pin-connected Pratt deck spans of 177 ft. ½-in., with a depth of 32 ft. The clearance is 50 ft. above low water and 20 ft. above high water. The steel work weighs about 960 tons, and was erected by means of a derrick car supported at the rail level by a temporary bridge built to carry trains and to support the steel superstructure during erection. The trestle approaches are 415 ft. long at the east end and 135 ft. at the west end. The Canadian Bridge Co., of Walkerville, Ont., had the contract for the steel work and its erection. The foundations and piers were built by Mackenzie, Mann & Co., of Toronto, the general contractors for the railway. Work on the substructure was commenced Aug. 15, 1903, and the erection of steel on Dec. 5, 1904. The bridge was completed March 18, 1905.

2. BRIDGE AT ELBOW. (First Crossing of the North Branch, 515 miles from Winnipeg.)—This bridge is designed for the same engine loading as the above, but with a train load of 4,500 lbs. There are eight spans of Pratt pin-connected trusses 157 ft. 9 ins. long and 28 ft. deep. The trestle approaches are 520 ft. long at the east end and 1,475 ft. at the west end. The bridge was completed Oct. 7, 1905.

3. BRIDGE NEAR BATTLEFORD. (Second Crossing of the North Branch, 578 miles from Winnipeg.)—This bridge has 11 spans of 166 ft. 6 ins. The weight of the steel was about 1,650 tons. The superstructure was erected by means of a traveler. Trains are carried on a temporary pile bridge 50 ft. from the main line. The Canada Foundry Co., of Toronto, has the contract for the steel work and erection, while the foundation and piers were built by Mackenzie, Mann & Co. Work on the substructure was commenced April 29, 1905, and the steel erection was commenced Sept. 23, 1905. It is expected that the bridge will be completed early this year.

4. BRIDGE AT FORT SASKATCHEWAN. (Third Crossing of the North Branch, 810 miles from Winnipeg.)—This bridge is for both railway and highway traffic; it has four deck truss spans of 186 ft. 1 in., and two 65-ft. girder spans. The live load on the trusses consists of two consolidation engines of 311,000 lbs., and a train of 4,500 lbs. per lin. ft. (the same as in the Battleford bridge), together with 1,200 lbs. per lin. ft. and a 1,600-lb. roadbed for the highway. The dead load is 3,450 lbs. per lin. ft. for the truss spans and 2,000 lbs. per lin. ft. for the girder spans. Fig 3 shows the tracklaying on the temporary timber bridge. The trestle approaches are 150 ft. long at the east end and 1,475 ft. at the west end. The Canada Foundry Co., of Toronto, has the contract for the steel work and erection. The foundations and piers were built by May & Sharp, of Edmonton. It is expected that the erection of the superstructure will be completed by April, 1906.

THE DRAINAGE OF THE FLORIDA EVERGLADES.*

By S. L. Lupfer.†

When, in the fall of 1904, Napoleon B. Broward was elected Governor of Florida, he renewed his campaign pledges, in which he had promised to do all he could to reclaim and open up the Everglades to settlement, and declared that if it were possible he would be "making the dirt fly" in the Everglades within a year after his inauguration.

When he took charge of the State Government, in January, 1905, and by virtue of his office became Chairman of the Board of Trustees of the Internal Improvement Fund, he found on hand in that fund about $352,000. This money had been accumulating for some time from the sale of lands belonging to that fund, and, according to the act of Congress of Sept. 28, 1850, which granted these lands to the State, the proceeds arising from the sale of these lands should "be used exclusively, as far as necessary, for their own reclamation and drainage." The Board of Trustees, which consists of the Governor, State Comptroller, Treasurer, Attorney General and Commissioner of Agriculture, were all in hearty accord with Governor Broward's ideas in regard to the reclamation of the Everglades.

The board was advised by its attorneys that the $352,000 belonging to the Internal Improvement Fund could be used for the drainage of the "swamp and overflowed" lands—which embraced the Everglades—and, in addition to this, the Legislature of 1905 passed a "drainage act," thus insuring the board sufficient money to carry out its ideas for the beginning of drainage operations on some well-defined plan.

After consulting a number of engineers and taking a trip around the borders of the section he wished to reclaim, Governor Broward, who is a practical mechanical engineer, decided to use dipper dredges, and after receiving bids from a number of dredge builders the Board of Trustees awarded a contract to the Featherstone Foundry & Machine Co., of Chicago, to build the machinery for two dredges.

The hulls of these dredges are to be 105 ft. long, 38 ft. beam and 7 ft. deep, and all the machinery will be placed above deck. One dredge will have a boiler of Scotch pattern, the other of locomotive type, each of 150 HP. A double 14 × 20-in. hoisting engine and a double 8 × 12-in. swinging engine will handle the 4½ cu. yd. dipper with which each dredge will be equipped. A special feature of these dredges will be the booms, which are very strongly built and 70 ft. long. They are so designed that the excavated material may be dropped as far back from the edge of the canal bank as possible. Most of the cutting will be through soft mud, and if there is not a wide berme the entire bank is liable to slide into the canal when the excavated material is piled on top of it. The depth of the boom at the center of dipper handle shaft is 84 ins., 12 ins. at outboard point and 18 ins. at the end of the angles and at heel castings.

The turntable, which is placed on top of truss, is 20 ft. in diameter. The A-frame is of structural steel construction and is about 45 ft. long on each side, from center of heel pin to center of bridle at the top. The two side spuds are 24 ins. square and the rear one 16 ins. For both the hoisting and swinging cables 1⅜-in. wire rope of the best quality is used, while a 1⅛-in. best Ulster iron short-link dredge chain is used for backing.

The main dredge engines are of the standard pattern used by the Featherstone Co., and all gearing is of cast steel, 2½-in. pitch and 7-in. face; main pinion is 15¼ ins., hoisting gear 83-in. and backing gear 47 ins. at pitch line.

LOWERING OF LAKE OKEECHOBEE.

While the board was struggling as to what style dredges it should use, the various members were "lying awake of nights" trying to determine which was the most practicable route by which the waters of Lake Okeechobee could be lowered.

This is the Queen of Lakes, covering over 1,200 sq. miles and being the largest lake lying wholly within the borders of one State. In order to drain the Everglades, which surround Lake Okeechobee and stretch to the south of it for over 100 miles, it would be necessary to permanently lower the level of the lake six or seven feet and retain it at that level, so that the marsh lands which surround it could drain into it and this water be carried to the sea through a system of canals.

It is questionable whether, without lowering Okeechobee too much, it will ever be possible to make the lands which lie nearest to the lake entirely free from overflow. It will probably be necessary to have each plantation dyked and a drainage pump ready, so that during an extraordinary rainy season, when the lake becomes very high, the water level of the plantation ditches can be kept down and the growing crops suffer no harm. The drainage pumps might not be needed every year, or not over a month in each year, yet when needed they would be needed badly, and the cost of their installation and maintenance would be a mere bagatelle in comparison with the good they would do.

Okeechobee, while not a deep lake, will average 15 to 18 ft. in depth a half mile from shore, so that the lowering of its waters 6 or 7 ft. would not greatly reduce its area, thus affecting the climate to the south of it; but to lower it much more than this and retain it at that level would invite the greatest danger to which these lands are subject—fire. All of the marsh lands surrounding Lake Okeechobee are composed of "peat," very similar to the world-famed Irish

*See also article by Mr. Lupfer on the same subject in Engineering News of Sept. 14, 1905.
†Kissimmee, Fla.

fuel, and when these marshes become thoroughly dry they burn with little flame but intense heat. As long as the water level is within 4 or 5 ft. of the surface there is no danger of the land burning to any great extent; and, as this question of fire is a well understood one, care will be taken not to lower the waters of Okeechobee sufficiently to invite this danger.

DIFFICULTIES OF SURVEYING THE EVERGLADES.

The question as to the proper route by which the waters of Lake Okeechobee be carried to the sea has always been a mooted one, and one great reason for this is that it is almost impracticable to make a survey of the southern routes. The report of U. S. Asst. Engineer J. W. Sackett will give some idea of the difficulty of making a survey of the section between Lake Okeechobee and Fort Thompson, the headwaters of the lower Caloosahatchee River.

Mr. Sackett says in part:

From information received from the captain of the sloop it was thought that a few miles north, it would be possible to get through the sawgrass and start the levels in close proximity to the lake. I took a portion of the party and sailed up the lake, with this object in view, and finally started through the sawgrass in what appeared to be the most promising place. After an exhausting struggle we succeeded in getting through; but it was found to be too far north to answer the purpose.

The Okeechobee Drainage Company's (Disston's) canal made access to Lake Okeechobee easy, but no place sufficiently stable to set an instrument was found along it. For a distance of 10½ miles from the lake no direct communication could be made between the waters of the river and the prairie.

This portion was surveyed in the following manner: Having brought a supply of pine strips, 3 ins. wide and 1 in. thick, 18 of them were nailed together in pairs in such a manner as to present the greatest amount of resistance to flexure, one end was sharpened, and they were used as stakes to build temporary stations in the canal near alternate sides and about 1,000 ft. apart. Both the level and the transit would fit the tripod belonging to either instrument. One skiff with two men was assigned to build the stations, which was simply to drive the stakes in the form of a tripod, leaving just enough room for a skiff to get between it and the sawgrass on the side of the canal or river, to lash the tops firmly together with a becket prepared for the purpose and to lash the instrument tripod to the station. They would then move ahead with the material for another station, which being erected, an observer, meanwhile, having placed the transit on the first station, took the magnetic bearing of the new station and read the stadia rod, which with the level-rod and sounding pole, was carried in the third boat, located at a point midway between the stations. Removing the transit he replaced it with the level. The level rod was held on the same stake as the stadia-rod, the stake being driven flush with the surface of the water.

As a check on the compass course, at each station an angle was taken to a solitary cypress tree, which formed a station near the Okeechobee end of the canal, and a short stake was left to mark the center of each station so as to be visible from the succeeding station. There being enough material for three stations, when the observer was through he moved the instrument tripod and pulled up the stakes. The party in the head skiff had by this time erected the third station and returned to the second, and the first set of stakes and the instrument tripod were transferred to it.

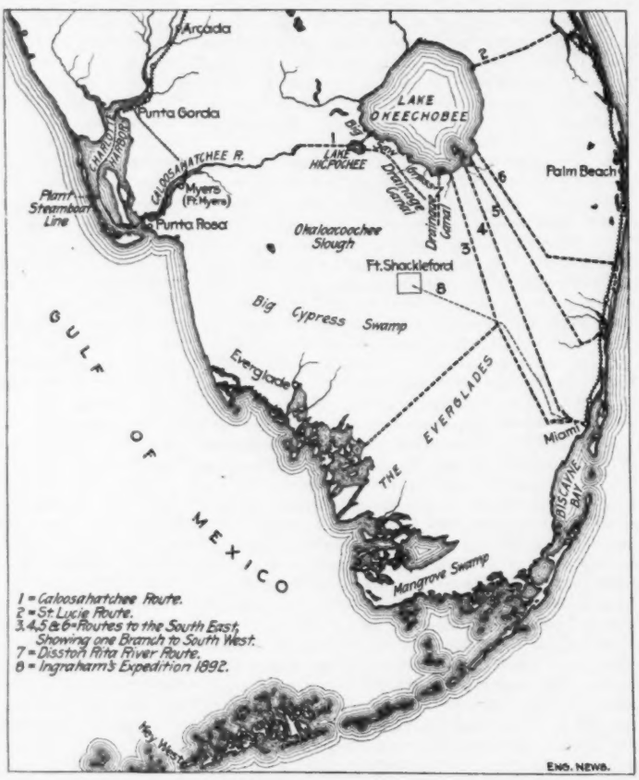
The instrument tripod was then placed on the third station. While the observer occupied the second station, the head skiff party proceeded to erect the fourth, etc. Under favorable circumstances a progress of about three miles a day could be made.

If the difficulty is so great where there is plenty of water to float a boat and an open waterway, one can easily see that it would be almost impossible to make an accurate survey to the southward, where there is nothing but crooked, shallow lagoons, most of which are so narrow and tortuous that they are only passable during high water, and even then most of them are completely overgrown with a species of sawgrass which will cut one's hand to the bone if handled the wrong way.

This sawgrass frequently grows to a height of 8 or 10 ft., and the greater part of the Everglades for the first 40 miles south of Lake Okeechobee is covered with it and a bush called the "custard apple," which forms a jungle that is almost impenetrable. It is impossible to make a survey except in boats, as every step a man takes he sinks to his knees in the mud and water, and in some places, if he goes overboard, he has trouble in extricating himself.

THE INGRAHAM EXPEDITION.

Probably the best organized expedition that ever attempted to cross the Everglades was the one under the leadership of Col. J. E. Ingraham, now Vice-President of the Florida East Coast Ry. This expedition left Fort Shackleford, about 25 miles to the southwest of Lake Okeechobee, on March 21, 1892, attempting to run a direct line to Miami, and the records of the trip show an amount of toil, privation and endurance that would do credit to the old pioneer days. For the first few days levels were taken and the line was chained, but as the main body of the Everglades was reached surveying had to be given up, and it became a serious question as to whether the party would ever reach solid land again. A few extracts from their records, as their secretary jotted them down, will give some idea as to the privations endured.



MAP OF SOUTH FLORIDA, SHOWING ROUTES PROPOSED FOR EVERGLADES DRAINAGE CANALS.

March 25. This was one of the hardest days yet—water scarce, sawgrass plentiful. Made one portage to-day of over 2,000 ft. Abandoned the smallest of our wooden boats and threw away some of our impediments before crossing.

March 27. Our flour is out. Corn meal has been out several days.

March. 28. Our consumption of food has heretofore been rather lavish and based on an expected average advance of 5 miles per day through the glades, which expectation has not been realized, and this step (short rations) has been taken none too soon.

This has been a terrible strain on everybody. Locomotion is difficult and slow. The bog is fearful and it sometimes seems as if it would be easier to stay in it than to go on.

The chainmen giving out—occasioned by physical exhaustion going through the high sawgrass, and limited food—chaining had to be given up for the present.

March 31. Made only two short portages through the sawgrass. This was very fortunate as the men are becoming quite weak and somewhat discouraged.

April 3. Were it not for the boats to lean upon, locomotion to-day would be extremely difficult if not impossible. Some of the deepest bogs yet encountered impeded our progress. The men walked beside the boats and as they bogged, raised themselves, and so continued. Our bacon has been low for some days but we still have coffee, tea and grits.

April 4. The constant wading in water and bog appears to have weakened all to a greater or less extent and considerable lassitude prevails, although no one is ill and the weather is perfect.

How they would have reached solid land is hard to say had they not on April 4, while still 25 miles out in the Glades, luckily fallen in with an Indian, who took several of them to Miami, returned with provisions for the others, and on April 7 arrived at Miami with the rest of the party.

A QUESTION OF ROUTES.

Engineers have been at work since last August to determine which is the most practicable route to start with a canal, and, as the board is very anxious to make no mistakes, it will probably be some time yet before the route is determined on; but in the meantime it will not be out of place to describe the four different suggested routes and make a prophecy that the last one described—that to the southeast—will eventually be chosen.

THE CALOOSAHATCHEE ROUTE.—The route which appeals to the engineer as the most feasible one by which Lake Okeechobee can be lowered is through the Caloosahatchee River. This route starts from the southwestern corner of Lake Okeechobee and runs west 2½ miles, traversing a deep sawgrass marsh, through which the Disston people cut a 6 × 25-ft. canal, into Lake Hikpochee, with a fall of 1.5 ft. Crossing this lake (4 miles), we again enter one of the Disston canals, 8 × 70 ft., and follow that for several miles, when the channel of the upper Caloosahatchee River is reached and the canal stops.

Following down the upper Caloosahatchee, we pass through Bonnett Lake, then through a short canal into Lake Flirt, out of which the Caloosahatchee River proper flows. Assuming that the elevation of Lake Okeechobee is 21 ft. above tidewater, which is the average figures given by the various surveys, we find that in the 22 miles from Lake Okeechobee to Fort Thompson, eight of which are through Lakes Hikpochee and Flirt, the fall is 14 ft., a foot per mile, as the canals and river are only 14 miles long, and this distance could be shortened a mile by various cut-offs.

The excavation on this route would be very easy, as it would be mostly muck for the first ten miles after leaving Lake Okeechobee, and the remainder of the way muck and sand, until Fort Thompson was reached, when a rotten limestone reef would have to be cut through in order to give a good outlet into the lower Caloosahatchee River.

So far, this is the route which any engineer would choose; but at Fort Thompson he would have to stop, as the formation of the Caloosahatchee River for 40 miles below that point would preclude turning the waters of Lake Okeechobee into it. This river has a bluff rock bank from 8 to 10 ft. high, and is so crooked that at some of the bends a 60-ft. steamboat has to be warped around.

Having a fall of only 7 ft. in the 40 miles between Fort Thompson and tidewater, with a depth of from 8 to 10 ft., a width of from 60 to 90 ft., and being so tortuous, it will readily be seen that this river can carry very little water, and when Lake Okeechobee rises 6 or 8 ins. and overflows the marshes spreading into the headwaters of this river the river itself rises 8 or 10 ft. When, as it does occasionally, the big lake rises 12 or 15 ins., the river for the first 25 miles below Fort Thompson rises 18 to 22 ft., inundating the entire country, ruining hundreds of acres of fine orange property and leaving destruction and desolation in its wake.

The question of straightening the river has been discussed on several occasions, but the fact of the river's bed being so small, and all the necessary excavation having to be made through solid rock, has disheartened all who investigated the matter.

Several years ago the citizens of the Caloosahatchee Valley, having secured permission of the State authorities, who assumed no responsibility, built a dam across the canal leading west from Lake Hikpochee, hoping thus to avert the overflow of their lands; but the first freshet which followed convinced them of the futility of this

plan, and when the United States Government condemned the dam, pronouncing it a bar to navigation, those who had built it removed it.

Had they gone four miles farther west to Sugarberry Hammock, and there built a levee, from highland to highland, strong enough to hold 5 or 6 ft. of water, leaving a narrow channel for the stream and making the river ends safe from washing, they would have forced the water south from Lake Okeechobee, and the Caloosahatchee Valley would have been safe from any great overflow; but until a great part of Okeechobee's drainage is diverted to the southward this valley must frequently suffer.

THE ST. LUCIE ROUTE.—Another route which has received favorable mention is the St. Lucie route. This leaves the northeastern corner of Lake Okeechobee and runs nearly due east to the headwaters of the St. Lucie River. This canal would be only 24 miles long, but as there would be an elevated plateau to cut through, and the excavation would be mostly sand, marl and considerable rock, the work would be enormously expensive. In addition to the great cost, the land through which this cut would be made is a pine barren and would not be benefited by the canal, while on the other routes suggested the land of all the sections penetrated by the canals is of such a character as to be greatly enhanced in value.

THE RITA RIVER ROUTE.—The next to consider is the Rita River route, and it was this one which the Disston Co. selected, but when they had reached a point 10 or 12 miles south of Okeechobee with an 8 × 50-ft. canal, they claimed that they had fulfilled their contract with the State and suspended operations. The surveys made by the Disston Co.'s engineers showed that in the first 24 miles south of Okeechobee they secured a fall of 12 ft., and their estimate was that by starting at Okeechobee and constructing a canal 10 × 150 ft., gradually excavating shallower until at the end of the 24 miles they would only be cutting 6 ft. deep; or, if they constructed two or more canals, whose combined cross-section equalled the 10 × 150-ft. canal, they could lower the lake enough to make the land surrounding it safe from overflow.

While in a dry season this canal would have lowered Okeechobee considerably, it is very doubtful whether this would have been protection enough to warrant the planting of these lands, for two reasons—first, these marsh lands will in the first few years of their being drained and cultivated, solidify, become compact and settle down at least 2 ft. from their present level; second, on this land, growing crops, and sugarcane especially, must have at least between 3 and 4 ft. drainage, or the land will become too sour for them to thrive.

This would allow a cross-section of only 5 × 150 ft. for drainage ere the danger line was reached, with a fall of 7 ft. in the 24 miles and a 6-ft. block in the outlet of the canal. Taking into account the immense rainfall which Florida has during the summer months and the area which drains into Okeechobee, it is very much to be doubted whether this canal could keep the lake from overflowing all the marshes again.

That this is one of the proper routes on which to cut a canal is unquestionably true, but it should be 12 ft. deep at Okeechobee and run 40 or 50 miles farther south than the Disston Co.'s engineers' estimates, where a tidewater outlet could be reached; and this should be only one of several canals leading to the southward.

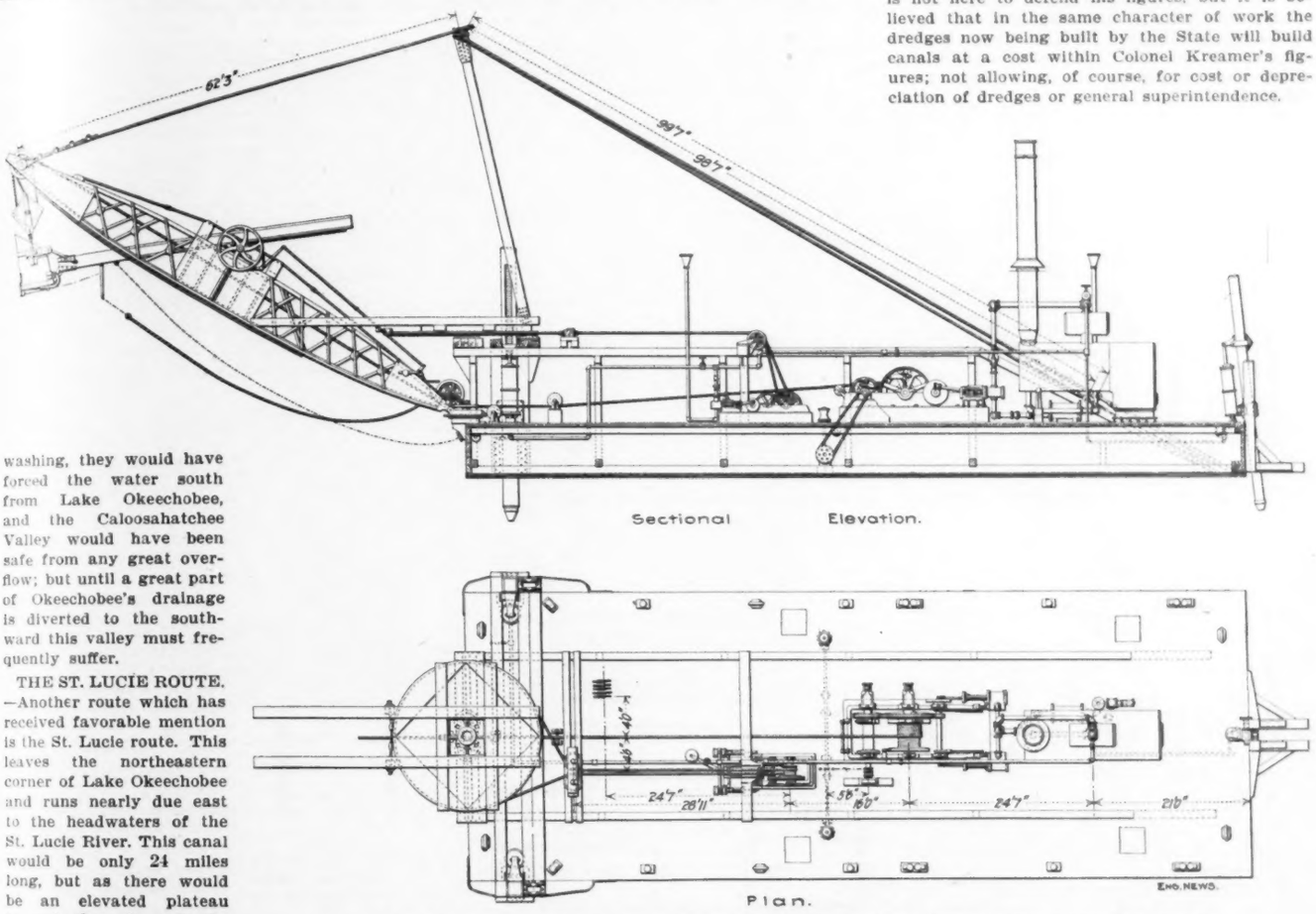
At the present ending of the Rita River canal there are rocks quite near the surface. Had this canal left the lake a few miles to the eastward and been cut in a south-southeasterly direction until past Fort Shackleford and then turned to the southwest into one of the rivers which empty into the Gulf in the vicinity of the Ten Thousand Islands, it would have given a much better current to the canal and the good marsh land to the south and east of Fort Shackleford would have become available for agricultural purposes.

As this canal would be nearly all muck excavation, it could be constructed very cheaply per mile. The late Col. J. M. Kreamer, Chief Engineer in charge of the Disston operations, said in regard to the cost of excavating:

> The canal proper already completed from Rita River is 50 ft. wide. Including labor, fuel, repairs and all extraordinary charges due to breakdowns and so forth the present canal cost an average of 1.6 cts. per cu. yd. This does not include cost of dredges or general superintendence.

While this price seems to be absurdly low to anyone who knows anything of dredging, it must be remembered that this canal was cut through a morass of a consistency considerably softer than cheese and of such a compressible character that a dipper of 4 cu. yds. capacity would probably bring up at one load the material which had occupied 5 or 6 cu. yds. in its original bed.

Colonel Kreamer has passed to his reward, and is not here to defend his figures, but it is believed that in the same character of work the dredges now being built by the State will build canals at a cost within Colonel Kreamer's figures; not allowing, of course, for cost or depreciation of dredges or general superintendence.



4½-CU. YD. DIPPER DREDGE WITH 70-FT. BOOM FOR FLORIDA EVERGLADES DRAINAGE CANAL.
The Featherstone Foundry & Machinery Co., Chicago, Builders.

THE NEW RIVER ROUTE.—Now, as to the fourth and last proposed route—to the southeast. Starting from Pelican Lake, which lies just to the southeast of Lake Okeechobee, and is connected with the big lake by a good-sized river, and heading in the direction of the Hillsboro, New and Miami rivers, no natural obstacles present themselves. All the way is a sawgrass marsh, interspersed near the lake with custard apple swamps, and farther out in the Glades are willow thickets, neither of which would present any difficulty to the dredges which will be used. The greater part of the way the muck is very deep; there would be no excavation necessary above the present water level; neither would rock be encountered until near the heads of the various rivers. The distance would be greater than by the two routes first spoken of, but the ease of excavation would overbalance that, and the land which the canals penetrated would be of the quality for which the drainage is being done, viz.: valuable sugar land.

How many canals will be necessary to reclaim the Everglades is a question which only time and experience can determine; but with four canals, each 12 × 75 ft., running from Lake Okeechobee to Hillsboro, New, Miami and Shark rivers, and one into the Caloosahatchee, it is very questionable whether Okeechobee would ever again rise to its present level; and with a system of cross canals connecting the main drainage ones every few miles the present morass would soon become solid land and the canals could be used to transport the products of this section to the sea, thus

providing the sugar industry, which will be the leading one, with the cheapest transportation to be had.

Although some of the land will become available within a year after the dredges start to work, the young men of to-day will probably be middle-aged before the last million yards of excavation necessary is made and the last canal opened for drainage and navigation. But if this work does require a long time and vast outlay of money, it will be a good investment, as the writers of years hence will place the drainage of the Everglades among the "big" works of the American people.

Since the above article was prepared the Board of Trustees has decided upon the New River route, and the erection of the dredges is now in progress at Fort Lauderdale, where the Florida East Coast R. R. crosses New River. This is a narrow stream, but averages 12 to 15 ft. in depth up to the edge of the Everglades. Here nearly a half-mile of rotten limestone must be cut through, and the canal will then be in a great sawgrass marsh with 10 ft. of muck. Of the $350,000 now on hand, $150,000 is allowed for the building of the dredges, a tender and four small barges, which will leave in the treasury $200,000 to carry on the work. In addition to this, the taxes on the land included in the drainage district will furnish $100,000 to $125,000 per annum for the prosecution of the work. It is hoped that within a year after the dredging begins some of the State lands will be reclaimed from overflow and can be sold to furnish further funds to prosecute the work.

[Since the above contribution was written and put in type, an injunction has been issued by Judge Locke, of the United States Court, restraining the State authorities from the collection of the special land tax in the drainage district.

The principal grounds for the decision are:

First—That no hearing was granted the property owners by the Drainage Act.

Second—That the trustees failed to tax the State land as well as the private land.

Third—The Legislature had no power to delegate its legislative powers to an administrative board of drainage commissioners. (Should the constitutional amendment, to be voted on in November next, be adopted, this ground would be eliminated.)

This decision will necessarily hamper the continued prosecution of the work until the Board of Trustees arranges some other plan for financing it.—Ed. Eng. News.]

GOVERNMENT TESTS OF FUELS AND STRUCTURAL MATERIALS.

The U. S. Geological Survey is asking for appropriations of $250,000 and $100,000, respectively, for the continuance (during the fiscal year beginning in June) of the systematic tests of fuels and of structural materials which were inaugurated in connection with the St. Louis Exhibition of 1904, and we give herewith an abstract of a report on this work made in February to the Secretary of the Interior by Mr. Charles D. Walcott, Director of the U. S. Geological Survey:

INVESTIGATIONS OF FUELS.—For the purpose of investigating the fuels of the United States, Congress appropriated during the year 1904 (in three acts), sums aggregating $85,000. The act of Congress making appropriations for the sundry civil expenses for the fiscal year ending June 30, 1906, carried an item of $202,000 "For the analyzing and testing of the coals, lignites, and other fuel substances of the United States, in order to determine their fuel values, and so forth, under the supervision of the Director of the U. S. Geological Survey." During the current fiscal year these investigations are being conducted at St. Louis, on the same site that was used during 1904. The equipment has been enlarged by the purchase of necessary instruments and machinery and by the construction of storage bins, conveyors, and other facilities for the handling of coal.

The samples of coals and lignites for these investigations have been and are being collected with great care by men trained for this purpose, who visit the different coal fields. The samples brought to the testing plant are analyzed and tested as to their suitableness for use under the steam boiler, in the gas producer, and in coke ovens—as to how far their quality may be improved by washing, and as to the ease with which they may be briquetted for fuel purposes.

The carefully made analyses of these representative coals and lignites are giving to the engineers of the country data of great value, not only because this work is done with care, but also because the complete history of every such sample is indicated, and the record is authentic and reliable. More than 1,600 such chemical analyses have been made, involving more than 7,000 chemical determinations. The following tests have also been made: In the gas producer and gas engine, 85 tests (each extending over two to three days) of coals, lignites, and peats from 18 States. In the steam boilers, 283 tests (each of 9 to 11 hours' duration) of 170 coals from 18 States. In the coke ovens, 110 tests (each of 48 to 72 hours' duration) of 95 coals from 16 States. Of these coals a large portion have heretofore been considered as non-coking coals.

In the washery plant it has been shown how the quality of 43 coals from 13 States and several samples of coke breeze could be greatly improved by washing, at a nominal cost of from 3 to 10 cts. per ton. In the briquetting plant a number of new possible binding materials for coal briquettes have been investigated, and a few of these have proved sufficiently satisfactory to warrant their more extended use, at a cost considerably less than that of the pitch, which is ordinarily used for briquetting operations. A number of lignites, representing both the extreme Northwest and the Southwest, have been briquetted under high pressure without the use of binding materials, and it has been shown that the cost of briquetting such materials should not exceed 50 cts. per ton. Furthermore, it has been shown that in the use of certain briquetted coals for railway and domestic uses the increase in the efficiency of the coals used in the briquetted form is more than sufficient to cover the cost of the briquetting operations. Since these tests were begun, two briquetting plants have been installed in California, and new briquetting equipment is now being tested in New York, St. Louis, Pittsburg, and Chicago.

The power-gas investigations have shown the suitability of the different American bituminous coals, lignites,

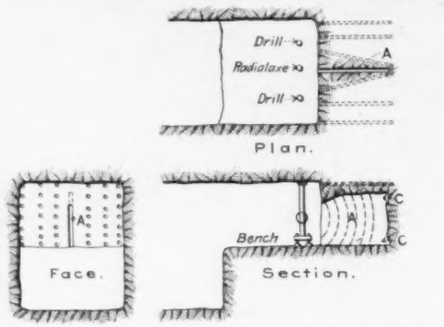


Fig. 1. Sketch Showing Method of Driving Rock Tunnel Heading with Drills and "Radialaxe" Channelers.

and peats for power purposes in the gas producer and the gas engine.

INVESTIGATION OF STRUCTURAL MATERIALS.—During 1902, 1903, and 1904 the survey conducted a series of examinations of materials suitable for the manufacture of cements found in different parts of the country, and these examinations have already resulted in important industrial developments.

Early in 1905 Congress made two small appropriations, aggregating $12,500, for the continuation and completion, at St. Louis, of certain investigations in structural materials (especially cement mortars and concretes), which investigations had been begun during the exposition at St. Louis.

The investigations now pending include the following:

(1) The examination as to nature and extent of the deposits of materials suitable for building and construction purposes, which are largely used at or near the important building centers of the country;

(2) The testing at the laboratories at St. Louis of the building materials collected by the survey experts in different part of the country; these include sands, gravels and stones (in massive, crushed, and pulverized form), all of which will be tested as to weight, strength, durability, power of absorption, behavior under heat and cold, and other features illustrating adaptability for use as building materials;

(3) The testing of cement mortars made with the different grades of sand and gravel, stone screenings, etc., as to the relative proportions of these materials which yield the best results in the mortar, and the different degrees of fineness, etc., in which these materials can be used to the best advantage for increasing the strength, compactness, and other desirable qualities of building material;

(4) The influence of time, temperature, and other variable conditions upon the setting, strength, porosity, permeability, and other properties of the cements and cement mortars;

(5) The tests of concretes made of cement with stone, stone screenings, gravel, sand, cinders, slags, etc., which are collected from different parts of the country, as to their strength, permeability, action under freezing, fire, and other properties which concern the adaptation of these concretes for use as a building material.

In connection with these investigations, and in response to numerous requests from the engineers and architects of the country, considerable attention has been given to the collection of information concerning the tests of cements and concretes which, from time to time, have been made in other local laboratories in different parts of the country. This investigation has shown that many of those tests have been made with care and intelligence, but their value has been greatly limited by the fact that the tests were not accompanied by proper examinations in the field as to the nature and extent of the deposits of materials which were used in making the tests.

A NEW METHOD OF ROCK TUNNELING UNDER CITY STREETS.

By Frank Richards, M. Am. Soc. M. E.*

We are only at the beginning of the subway and tunneling era in and around New York and other large cities. The first subway of the Interborough Rapid Transit system is so great a success that the completed line is to be widely extended.

We see the beginning, but the end challenges our imagination. The system which is to provide rapid and convenient transportation for the great population whose homes and business places are many miles apart, and whose activities and interests demand frequent local trips between, will later be employed also for the transfer of freight and supplies to convenient points of distribution. The great railways terminating at New York must ont only be able to cross the city, but must be able to reach passenger stations centrally located, and construction work is already in active progress to satisfy these demands.

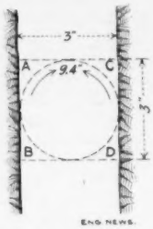


Fig. 2. Sketch Illustrating Relative Freedom of Fracture of "Radialaxe" Channeler and Ordinary Rock Drill.

But in underground excavation in great cities there must be stringent restrictions as to the conduct of the work. The operations must, as far as possible, be carried on without tearing up streets or interfering with surface traffic; and most of the future subways will be built by tunneling.

In New York most of the cutting must be in the solid rock, and for driving tunnels in the built up portions of the city, where buildings have deep foundations and the superstructures are high and costly, it is imperative that the work shall be carried on without damage or possibility of damage to these structures. Besides the foundations of heavy and towering structures which are to be respected, the various crossings of the lines present perhaps the greatest risk of all in city tunnel driving. A tunnel to be driven across the line of a tunnel already existing must necessarily go under it, and this its engineers will plan to do without going to a greater depth than may be compulsory. The new tunnel will therefore have a thin roof at the point of crossing and to cut along there without imperiling the line overhead will be a serious problem.

Rock drilling and blasting, must of course be employed in the tunnel driving, but these operations must be carried on under such restrictions that almost absolute safety will be guaranteed. The contractors' anxiety to advance as rapidly and at as low a cost as possible must be curbed by the enforcement of conditions to secure safety. If alert ingenuity devises new means and methods for securing safe conditions, they are not only to be welcomed but it would seem to be the proper thing to insist upon their employment.

*Ingersoll-Rand Co., New York City.

In rock tunnel work the most onerous part is the driving of the heading, and in the driving of the heading the most severe and trying operation is in making the first break or opening in the solid face of the rock. The usual practice is to drill two sets of nearly horizontal holes, starting at about 4 or 5 ft. apart and converging toward the center, and then firing all the holes at once to throw out the triangular prism of rock between vertical channel A. This channel has a depth or length forward extending into the face of, say, 8 ft. and a height or width vertically of, say, 5 ft. The thickness of the channel is about 2½ ins. at the front and about 1¾-ins. at the back. The thickness of the channel obviously makes no difference in the result. There it is, and when the lifting holes on either side are fired, the two sides being fired separately, the rock can give

FIG. 3. VIEW SHOWING "RADIALAXE" CHANNELING MACHINE IN OPERATION IN BELMONT TUNNELS, NEW YORK CITY.

them. This is, of course, effective where heavy charges of dynamite can be used as in ordinary railroad or mine tunnel work. Under city streets, however, where every charge of dynamite must be carefully limited and where the permissible amount to be exploded at once is in fact defined by municipal enactment, the case is as different as can be. In the driving of the subway tunnel in Park Ave., New York, not only were the foundations of houses seriously damaged by the dynamite explosions but windows were repeatedly broken by the outrush of air from the shaft impelled by suddenly released gases.

The reactive shock of the explosive in the first operation of lifting out the center of the heading is much more severe than splitting off successive portions from the then opened face, which is the subsequent operation in enlarging and completing the tunnel opening to its required section. In the first explosion there is great resistance to movement, and this resistance is only to be overcome by the use of much larger charges of the explosive.

It is obvious that if the first operation of breaking out the face for the advance of the heading could be transformed into the simple and easy operation of splitting off slabs of rock with nothing opposed to or resisting their movement away from the exploded charge, then the most severe shocks of explosion would be avoided. This desirable result may evidently be completely accomplished by cutting a central vertical channel in the heading, and this plan has been adopted in driving the two crosstown tunnels of the Pennsylvania R. R. in 32d and 33d Sts., New York, and also in the Belmont twin tunnels in 42d St., these latter constituting an easterly extension of the subway system and connecting through an East River tunnel with Long Island City.

The operation here referred to will be readily understood by reference to the sketch, Fig. 1. Into the rock face of the heading, as the beginning of another advance there is cut a central way towards the channel just as readily as if it had "all out doors" there. When the holes on the other side are fired the rock there gives way as easily as on the other, and after that the other holes can be fired with the usual satisfactory result.

One of the most important facts to be noted in this process is that the charge of dynamite required for the lifting holes is only one quarter the amount that would be required to make the required break without the channel. It will be noticed that the holes are drilled in beyond the end of the channel, experience having shown that this practice gives the best results.

The apparatus employed in this work is the "Radialaxe" channeling machine, built and controlled exclusively by the Ingersoll-Rand Co. The method of tunneling is patented by Mr. W. L. Saunders, M. Am. Soc. C. E., President of the Company. The general principle of the construction is already in extensive use on the "Radialaxe" coal cutter built by this same company. In coal cutting a special bit is used without any rotation device. In the "Radialaxe" channeling machine the regular cross bit is used as in ordinary drilling work and the drill or cutter has the regular feed and rotation common to the ordinary rock drill. The cutting engine is mounted upon a column with a quadrant and worm device for swinging the drill within the range required in a fixed vertical plane. This radial movement is secured by means of a hand wheel on the worm shaft and the drill is fed forward by means of an ordinary crank and feed screw on the mounting. The manipulation and control of the new machine is thus very simple and the operator is relieved from all shock and jar.

While no mathematical accuracy is required in placing the drill it is still essential that the column be well secured in its position, as a slight lateral movement at right angles to the channel after work has been fairly started would cause the steel to bind on the sides, giving constant trouble. If the column should actually move or fall so as to require resetting it would be difficult to so locate it again that the steel would find its normal and free movement. For this reason it is well to place the column close to the face, so that the steel may be as short as possible in starting; and the first cutting should be light, so that the channel may not at the beginning be crowded laterally by encountering hard or soft spots or flaws in the rock; for any irregularity at the beginning of the cut will, of course, simply increase with the depth.

With the drill fairly started in the cut, it is swung up and down by the quadrant and worm until it has traversed the entire angle required; at the same time it is gradually fed forward. The channel may in this way be made wider up or down or both as the drill advances into the cut. When the face of the rock is once fairly entered the full range of angular movement is used, this full angular travel may continue as long as desired and the drill will advance radially at the top and bottom, widening the channel as it goes deeper. To do this the drill is allowed to cut in at the top and bottom of the channel in advance of the cut, as shown by C in Fig. 1. When the channel is wide enough and it is desired to advance the top and bottom in approximately horizontal lines, the angular movement is gradually reduced as the channel cuts deeper. A good stream of water is kept constantly running into the channel and the chips falling to the bottom are easily removed with a hook without stopping the machine for clearing out.

A remarkable feature of the "Radialaxe" channeler is its high cutting speed. The rate of advance is much more rapid than that in drilling a hole of diameter equal to the channel width. This is due to two peculiarities of the cutting process as carried on by this machine.

First, the cuttings do not hamper its action. In drilling a hole, the chips loosened from the face remain at the bottom of the hole (except in an "up" hole) and are pounded by the bit, ground to a fine powder, and gradually worked back around the steel. The presence of these cuttings around the steel adds to the friction and resistance in the hole. Their presence at the bottom obstructs the face and prevents the bit securing the full cutting effect it would have if the face were clean. In the case of the "Radialaxe," however, the cuttings fall at once to the bottom of the channel and are removed with the scraper. The bit thus always strikes a clear face with maximum effect.

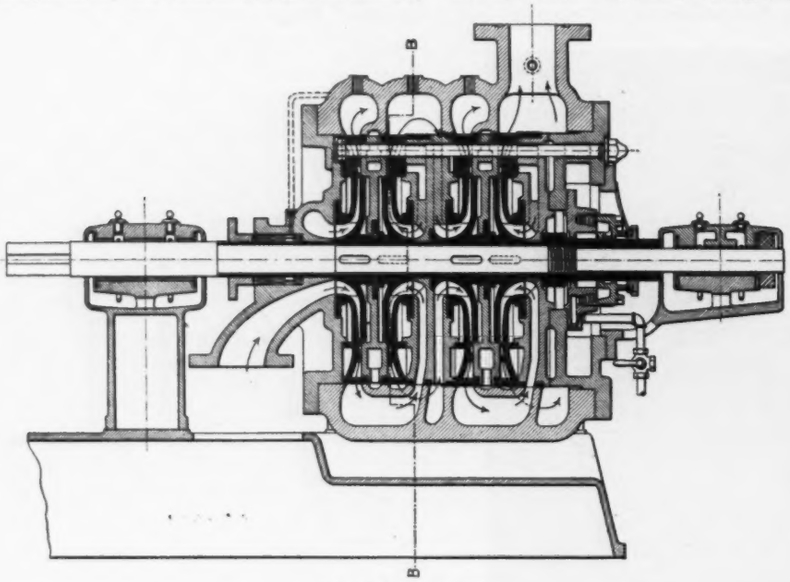
Second, the blow of the "Radialaxe" bit on the face is more effective in dislodging rock than the blow of a drill of the same size in a hole of diameter equivalent to the width of the channel face. This can best be understood by reference to the sketch, Fig. 2. The dotted circle represents a drill hole 3 ins. in diameter. In drilling this hole, the line shattering effect is limited to this circle; and the length of the line over which positive fracture must occur in order to loosen a chip over the entire area is the circumference of this surface or 9.4 ins. But suppose that the same bit is cutting in the channel shown by the solid lines, 3 ins. wide. The maximum length along which positive fracture must occur in this case is the sum of the two lines A B and C D, or 6 ins., less than two-thirds that in the former case. Transversely of the lines A C and B D, the fracture may extend upward and downward over the channel face. The result of this greater freedom of fracture is that larger chips are removed by the "Radialaxe" in its channel and its cutting capacity; or rate of advance is one-third to one-half greater than that of a drill of the same size in a hole of same diameter.

The result of these two distinctive features of "Radialaxe" channeling is that a channel such as is indicated in Fig. 1 is cut, in the average New York rock, in a working day of 8 hours, including all the necessary changes of steels.

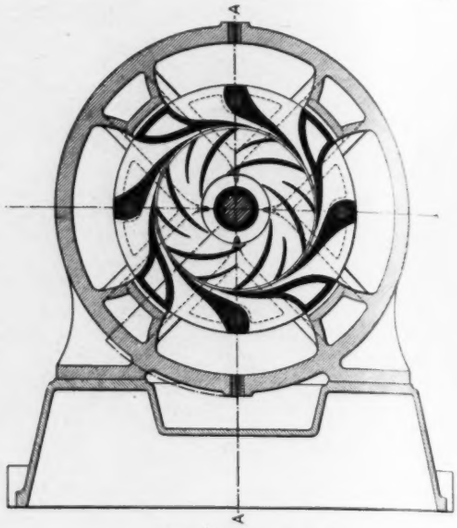
The set of steels used with the "Radialaxe" channeler usually advances by lengths of 6 ins., as it has been found that 6 ins. is about the maximum depth which a bit will cut without redressing. But this is in itself a remarkable performance considering the amount of rock which is

removed to a depth of 6 ins. over the full width and length of the channel.

The small space required by the channeler itself makes it possible to mount drills on either side of it and to drill side holes simultaneously with the cutting of the channel. It is to be noted that the actual lineal advance of the heading accomplished by this combined system of channeling and drilling is greater than where the channeling is not employed. There is no additional cost of operation involved except that of the "Radialaxe" channeler itself; and there is, as already noted, a saving in the amount of dynamite used.



Section A-A.



Section B-B.

FIGS. 1 AND 2. LONGITUDINAL AND TRANSVERSE SECTIONS OF FOUR-STAGE TURBINE PUMP; BUFFALO STEAM PUMP CO., BUFFALO, N. Y.

TURBINE PUMPS WITH BALANCED IMPELLERS.

The continuing commercial development of turbine pumps is illustrated by the accompanying illustrations and test results of two out of a line of turbine pumps recently brought out by the Buffalo Steam Pump Co., of Buffalo, N. Y., an offshoot of the long-established Buffalo Forge Co. The main feature of this line of machines is the use of back-to-back impellers (for approximate balance of thrust), supplemented by a carefully-designed arrangement of thrust-chambers

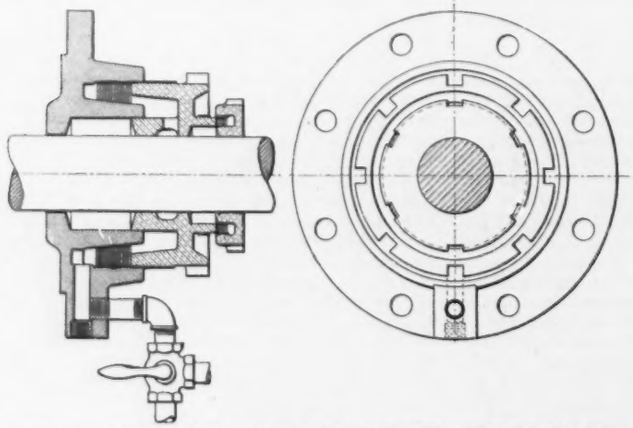


FIG. 3. DOUBLE STUFFING-BOX AT DELIVERY END OF SHAFT.

quite similar to those frequently used in single horizontal-shaft Francis turbines. Balancing multi-stage turbine pumps by the back-to-back arrangement is not a new principle in this branch of machine design,* but it has not found universal adoption. The fact that it makes more difficult the proper design of the water-passages (with regard both to construction and to operation of the pump) doubtless has something to do with this. The pumps here described, however, show water-passages of satisfactory simplicity and a general design that should prove cheap in machine work and assembling. Other important features of these pumps are, first, the separation of the functions of shaft-bearing and stuffing-box, and, second, the use of double stuffing-boxes to prevent leakage along the shaft.

Fig. 1, an axial section through a four-stage Buffalo turbine pump, shows the arrangement of the impellers or runners and of the water passages. The latter are more clearly explained in Fig. 2, a cross-section at right angles to the shaft. The pump consists of two main portions: (1) an outside cylindrical casing, in which are cored the discharge chambers of the several component turbines or "stages," and (2) within this casing an aggregation of plate-like guide castings, in which are formed the diffusion or guide vanes and the water-passages leading from the discharge chambers to the suction sides of the respective following units. Running in completely enclosed circular spaces between these guide sections are the runners. The aggregation of guide sections is assembled outside of the casing, with the runners or impellers put in place and keyed on the shaft as the assembling progresses, and is then bolted together by four through bolts in the manner shown in Fig. 1. The assemblage of working parts is set into the casing from one end, and is held in place by screws in the flange of the last section, which forms the end-plate of the pump case at the delivery end. In order to facilitate the work of placing it into the casing, the several ring-shaped seats on the internal surface of the case are of successively smaller diameter. At each of these seats the corresponding guide section contains a brass ring turned to a force-fit in its seat, but small enough to pass easily through all seats farther to the right (in Fig. 1). The assemblage is thus easily entered in the casing up to the short distance of force-fit, when all the brass rings are simultaneously pressed home to a tight fit.

The shaft bearings are wholly outside of the case. The main bearing (at the left in Fig. 1) is in a pedestal set on the base-plate, while the out-board bearing is formed in a bracket bolted to the end-plate of the casing. The ends of both bearings are fitted with babbitted shoulders, against which bear collars on the shaft, to regulate end-play and to take up any slight thrust not balanced within the pump. Where the shaft passes through the casing, stuffing-boxes are provided. Thus the bearings are not required to act as packings, and the packing is not required to carry load, an arrangement conducive to long life of both packing and bearings.

The stuffing-boxes are, as already noted, designed with special view to preventing leakage. They are double-packed, with two packings in series. The stuffing-box on the delivery side, whose construction is shown more in detail in Fig. 3, has two separate glands, the outer one being formed in the inner follower. The latter is so shaped as to form a leakage pocket between the two packings, and this pocket is drained by a drip-pipe; thus no pressure can accumulate in the space between the two packings, and the outer packing has to take care of only the moisture creeping along the shaft from the inner packing. The suction end stuffing-box is of somewhat simpler construction, having but one gland and one follower, with two packings kept apart by a separator. The space between the two packings is connected by a pipe to the delivery side of the first stage of the turbine, to form a water seal and prevent the leakage of air into the suction chamber.

Where the shaft passes through the stuffing-boxes it is protected by brass or composition sleeves, and a similar sleeve is placed over the section of shaft between the two pairs of impellers. This results in giving the shaft a complete non-corrosive covering from outside to outside of stuffing-boxes.

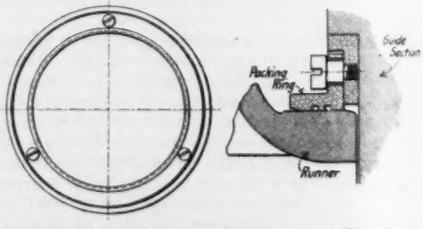


Fig. 4. Detail of Seal Ring for Thrust Chambers.

To balance the end thrust produced by the difference of pressures on the front and back of each impeller, thrust-chambers are formed between the outer faces of each impeller and the adjoining guide castings, in which water from the delivery side of the respective impeller accu-

*See, for example, the Sulzer high-head turbine pumps described in Eng. News, Jan. 23, 1902, p. 66.

mulates and produces a thrust proportionate to the exposed area of impeller face. These chambers are sealed off from each other by brass packing rings of special construction, which prevent the flow of water from the high to the low pressure spaces outside of the impellers.

Fig. 4 is a detail section of such a seal ring. The ring, which has a ground fit on the hub of the impeller, is held from rotating, but has a slight radial play, so that it can adjust itself to the eccentricity and vibration of the rotating parts. A brass seating ring is fixed in a recess in the face of the guide section, and the packing ring is held against this seat by three studs which pass through slotted holes in the packing ring. The long axis of these slots is radial to the shaft, so that the packing ring can shift slightly in all directions in a plane normal to the shaft. The brass seat against which the packing ring bears ensures a tight joint on the radial face, and the ground fit of the packing ring over the impeller hub prevents leakage and lose of pressure through the circumferential joint.

Each thrust-chamber, being in communication with the delivery side of its impeller by way of the radial clearance between impeller and guide vanes, contains water at the delivery pressure of that impeller. The two chambers produce thrusts in opposite direction, of an amount equal to their difference in area. This difference, in the present design, is so adjusted that for each pair of impellers the resultant thrust-chamber pressure just balances the difference of inlet opening pressures.

The balancing is thus accomplished separately for each pair of impellers, instead of providing a single balancing device for the entire pump, as is sometimes done. The method of thrust-chamber balancing is not restricted to pumps with back-to-back impellers, but can be applied just as well to any arrangement of runners. In fact, it is used also in the single-runner single-stage pumps of the same makers.

The makers report that these pumps have been built with full success for very high heads, even in sizes hitherto thought impracticably small for high head work. Pumps in capacities of 400 gallons per minute and less have been operated economically against heads as high as 1,500 ft. A recent high-head machine of this type is an eight-stage pump of capacity of 300 gals. per min. against 1,400 ft. head. This pump, shown in Fig. 5, really consists of two four-stage units in series, coupled in common to the shaft of an electric motor.

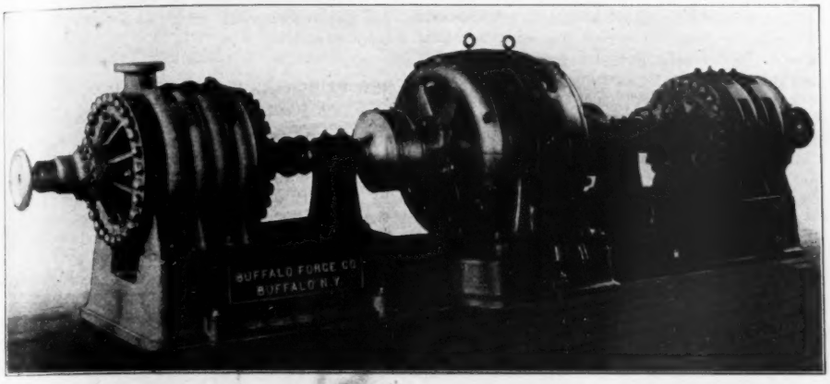


FIG. 5. VIEW OF EIGHT-STAGE BUFFALO TURBINE PUMP DESIGNED FOR 1,400-FT. HEAD.

Two tests made by the Buffalo Steam Pump Co. recently illustrate the performance of these turbine pumps (Figs. 6 and 7). The curves of Fig. 6 refer to a four-stage turbine pump designed to discharge 250 gals. per min. against a head of 630 ft. As the curves show, the specified delivery corresponds very closely both to the point of most efficient working and to the desired delivery-head. The efficiency under this condition is nearly 73%. The pump works at more than 70% efficiency over the range from 185 to 310 gals. per min. discharge, corresponding to a variation of 90 ft. in head. That is to say, a total variation in head of 14% of normal produces a variation in discharge amounting to 46% of normal, with practically the maximum efficiency throughout this range.

An efficiency only slightly lower than that of the preceding pump was shown by a pump tested with the results shown in Fig. 7. The maximum efficiency, 71%, was obtained at a discharge of about 1,000 gals. per min. against 40 ft. head. An efficiency of 70% or over is maintained through the range from 875 to 1,050 gals. per min., corresponding to a variation of 4.5 ft. in head. In this pump, therefore, a variation in the delivery head by 11% (of normal) varies the discharge by 17.5% of normal, while maintaining full efficiency. The latter pump thus has a steeper head-delivery characteristic than that of Fig. 6, although the scales used make it appear that the former has the steeper characteristic. The pump of Fig. 7 is much more nearly a constant-power machine than the other, which is corroborated by the power curves.

The fact that turbine pumps can be designed to have characteristic curves of any desired shape (within limits) is a very important element in their adaptability to different classes of work. For example, a pump intended for certain kinds of drainage work might be required to show a large increase in discharge for a small decrease in head, so as to keep the water at a nearly constant level. This result calls for a very flat head-delivery characteristic. Another pump might be required to have a practically constant load, for example, the full capacity of the driving motor. Such a pump would require its discharge to vary about inversely as the head, giving a characteristic quite different from that of the other case. Most such requirements can be met by careful design coupled with experience in the performance of completed pumps.

The shape of the impeller blade curves is the principal variable. Its relation to the performance of the pump is summarized in a general way by the makers in the following statement:

> The usual design of impeller has the blades turned away from the direction of rotation at a sharp angle, for the reason that this design allows a larger range of head and capacity with less falling off in efficiency. In frequent cases, however, the blades are radial as they approach the periphery of the impeller, or they may even be turned forward in the direction of rotation. This is common in the design of a pump the speed of which is limited by the engine or motor direct-connected to it. Generally speaking, the backward-turned blades are to be used wherever there is danger of overloading the motor by a decrease in head, and the radial or forward-turned impeller to obtain a greater head at a given speed, or for service which requires the head to be kept up to the normal through a considerable variation in the water quantity delivered, as for city water supply, boiler feeding or condenser circulating pumps. To a certain extent, however, any of these impellers may be so designed as to prevent serious overloading through a considerable range in head.

The Buffalo turbine pumps are made in a large range of sizes, from 4-in. to 15-in. regular, and larger to special order; the figures given refer to the diameter of the suction pipe. The corresponding capacities, at a flow-velocity of 10 ft. per sec., are 390 to 5,500 gals. per min. The speed of revolution is fixed according to the head.

As already noted these pumps are built single-stage or multi-stage. When the former, usually a double impeller is used and a double suction pipe, thereby giving complete balance as regards axial thrust; this refers to pumps with horizontal shaft. However, the method of balancing by thrust-chambers as above described makes it a simple matter to secure accurate balance even with a single impeller. In vertical-shaft pumps* the construction is such as will give a resultant upward thrust, to relieve the step-bearing pressure. Single-stage pumps are, usually, limited to 150 ft. head, or less. For higher heads multi-stage pumps are used, allowing from 100 ft. to (in extreme cases) 200 ft. head per stage. The highest degree of compounding represented in any pumps yet built by this company is eight-stage, but there appears to be no reason why more stages can not be used when required.

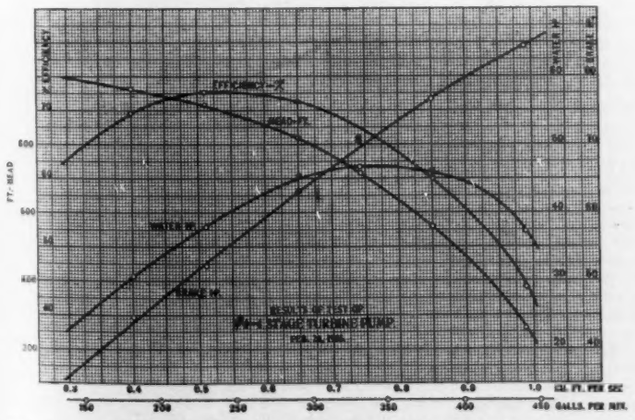


FIG. 6. CURVES OF RESULTS OF TESTS ON FOUR-STAGE BUFFALO TURBINE PUMP DESIGNED FOR 630-FT. HEAD.

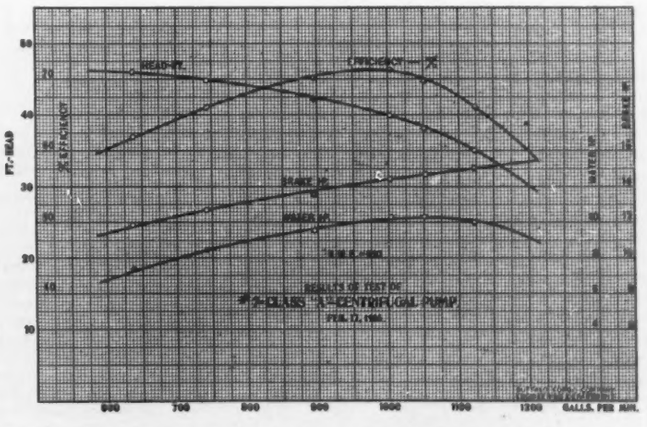


FIG. 7. CURVES OF RESULTS OF TESTS ON SINGLE-STAGE BUFFALO TURBINE PUMP DESIGNED FOR 40-FT. HEAD.

*See a description of a vertical Buffalo turbine-pump, single-stage, built for the water-works of La Salle, Ill., in Eng. News of June 30, 1904, p. 619.

A BELT CONVEYOR PLANT FOR HANDLING CONCRETE AND CONCRETE MATERIALS.

To handle the large yardage of concrete and concrete materials in constructing the foundations for the new gas works at Astoria, Long Island, use was made of belt conveyors for handling both the concrete materials to the mixers and the mixed concrete from the mixers. The principal details of this conveyor system are shown in the accompanying illustrations. The general arrangement is as follows:

The sand and gravel is brought to the work in barges, which are unloaded by a derrick equipped with a grab-bucket and discharging into a small hopper set over a service railway track. This hopper is of small size, for loading purposes only, and discharges by a bottom chute into dump cars on the track underneath. The cars deliver to a series of bins in which sand and gravel are stored separately. The receiving end of a Robins belt conveyor, under these bins, discharges onto it through bottom chutes. From the storage bins the conveyor extends 228 ft. to hoppers set near two concrete mixers. After passing through the mixers, the concrete falls onto a horizontal belt conveyor extending 50 ft. and discharging into cars on a service railway reaching to all parts of the construction work.

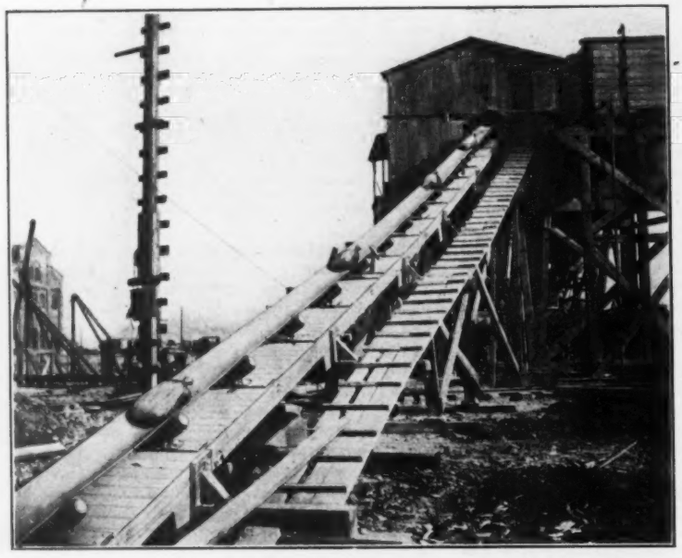


FIG. 1. VIEW OF UP-CURVE OF CONVEYOR DELIVERING CONCRETE MATERIALS FROM STORAGE BINS TO MIXER HOUSE.

The conveyor reaching from the storage bins to the hoppers over the mixers has a 20-in. belt. It runs horizontally for a distance of 105 ft. from the storage bins and then curves upward on a 250-ft. radius to the hoppers, which are 123 ft. further ahead on the horizontal. The curved portion of the conveyor is shown by the view Fig. 1. At the time of the photograph bags of cement were being handled. These bags are placed on the conveyor when it commences to curve upward and are delivered to storing platforms in the mixer house, also shown by Fig. 1. The belt was operated at a speed of 350 ft. per minute, and at that speed handled 100 cu. yds. of either sand or gravel per hour, this being the maximum that could be handled by the mixers, which, in fact, turned out about 35 cu. yds. each of concrete per hour. In respect to the construction of the conveyor, the only special feature to be noted is the employment of idlers equipped for internal grease lubrication. This was necessary because of the great amount of dust and grit which had, of course, to be kept out of the bearings.

Fig. 2 shows the arrangement of the various apparatus in the mixer house. The conveyor from the storage bins discharges into a two-compartment hopper a deflector being provided to turn the discharge into either hopper desired. From this hopper the sand and stone are chuted into the two mixers, set side by side in a line transversely to the conveyor line. Water and cement are added and the concrete mixed. A Y-shaped chute, the stem of which comes directly over a 24-in. conveyor belt and the arms of which, spaced right and left to positions under the discharge spouts of the two mixers, conveys the mixed concrete to the second conveyor. The arrangement is clearly shown by the drawings.

Fig. 3 is a view from the discharge end of the smaller conveyor, that takes the mixed concrete to the service cars. The concrete conveyor is 24 ins. wide and 50 ft. long between centers, and the belt operates at a speed of 400 ft. per minute. Practically the only feature of this conveyor that is special is the belt cleaning device. In order to prevent the concrete from adhering to the belt, a high-speed rotary cleaning brush, driven through gears and belt from the head shaft of the conveyor, is placed at the discharge end of the conveyor. This brush, revolving at a speed of 400 r. p. m., prevents the concrete from sticking to the belt and being pressed between it and the return idler pulleys. From the concrete conveyor the material is discharged through a chute 20 ft. long to cars, which are taken on service tracks to any point where work is progressing. These cars, when receiving their loads, stand on tracks about 15 ft. below the head end of the conveyor. Some difficulty was at first experienced because of inability to shift the cars fast enough to receive the material delivered to the conveyor without having a considerable quantity wasted by being spilled upon the ground, but this difficulty was overcome.

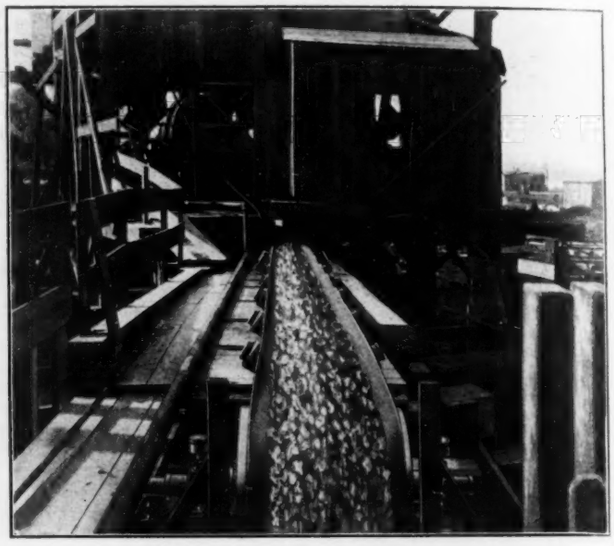


FIG. 3. VIEW FROM DISCHARGE END OF CONCRETE CONVEYOR FROM MIXER HOUSE TO CARS.

In general reference to the handling of concrete by belt conveyors, the builders say:

> The application of belt conveyors for handling fresh concrete is not entirely new, but it has only come into general use among contractors during the past year or two. Contractors felt that the service demanded was too great for the belt and that it would wear out in a short time. This has not proven to be the case, however, as is shown by the fact that the concrete conveyor belt is in as good, or possibly better, condition than is the belt of the conveyor used for carrying the concrete materials to the mixer. The Robins Conveying Belt Co. has furnished a number of these concrete conveyors for use around New York and in Philadelphia.
>
> A contractor's plant of the kind described in the foregoing is particularly adapted for the turning out of great quantities of concrete. As will be noted, the capacity of the conveyor system is limited only by the capacity of the mixers and by increasing the number of mixers it is possible to turn out a great increase in the amount of material, even with the same conveying machinery. Where contractors are confronted by a problem of this kind, it is quite often solved by an arrangement similar to this, but another arrangement by which the same results could be obtained is as follows:
>
> A storage bin for sand and gravel can be constructed with the conveyor running beneath the bins horizontally and at the end gradually curve upward until it reaches a height necessary for a system of gravity mixers. From these mixers, the concrete would be delivered directly to cars, wheel barrows or wagons without the need of a conveyor for handling the mixed concrete. This arrangement was employed by J. C. Rodgers & Co., of New York, in their work at the Norfolk Navy Yard, Norfolk, Va. The decision relative to the arrangement to be used, however, is affected to a large extent by the contractor's preference for gravity or rotary mixers.

The conveyor system described above was designed and built by the Robins Conveying Belt Co., New York, N. Y., to which we are indebted for the information from which this article has been prepared.

NEW PRINCIPLES IN THE VALUATION OF WELLS AS A PART OF WATER-WORKS PLANTS.

A unique estimate of the value of wells as sources of public water supply has recently been made in connection with the proposed purchase by the City of New York of the properties of the three water companies supplying the Borough of Richmond (Staten Island). The valuation of the properties complete was entrusted by the city to a commission of five members, consisting of Prof. William H. Burr and Messrs. I. M. de Varona, Nelson P. Lewis, Allen Hazen and J. Waldo Smith, all Members of the American Society of Civil Engineers. The data on which the estimates were based were collected largely by Mr. Richard Schermerhorn, Jr. Mr. A. S. Tuttle, M. Am. Soc. C. E., assisted in making the estimates, the details of which were entrusted to a subcommittee composed of Messrs. Lewis and Hazen. Special points bearing upon the value of the wells were that the water from some of them is bad, that the yield is insufficient, that a supply for Staten Island will probably be available within ten years from the Catskills, and that a contract providing for a temporary supply from New Jersey is proposed, but is held up on account of opposition to taking water from New Jersey for sale in New York. The estimated yield of the wells was placed at 750,000 gals. per square mile of tributary drainage area. The gross value of the water from the wells was then computed at the rate of $70 per 1,000,000 gals. Some of the water is very hard, the hardness for the half-dozen groups of wells ranging from 90 to 180 parts per 1,000,000. A deduction of 10 cts. per 1,000,000 gals. was made for each part of hardness over 50 per 1,000,000 gals. In the case of one group of wells a deduction of $10 per 1,000,000 gals. was made for high iron and salt contents, the latter being due to proximity of the wells to tide water. A further deduction of $20 per 1,000,000 gals. in one case, and $10 in another, was made on account of probable pollution due to the population on the collecting areas. The net re-

sult of the various deductions outlined was to make the prices for the several groups of wells $27, $47, and for five groups from $64 to $66 per 1,000,000 gals., in place of the original $70. The next step in the estimates was to deduct from the unit costs thus far reached the cost of pumping at the various stations. These deductions gave net unit values for the water, as delivered to the force mains, of $6.30 to $42.80. Multiplying these net unit values by the yearly yields at each station gave net annual incomes for each station. These figures discounted for ten years at 5% gave a present value 7.72 times the annual value. The final deduction, applying to some of the stations only, was the cost of new machinery necessary to keep up the supply for the full period of ten years. The valuation thus secured was on the assumption, as will be seen, that the wells themselves would be useless at the end of the ten years. There would, however, be a small salvage value in the station equipments and a good value in the land at the several stations. Approximations of these values were made by assuming that the entire value of the stations and equipment, and land, at the end of ten years would be twice the assessed valuation of the land for the year 1905.

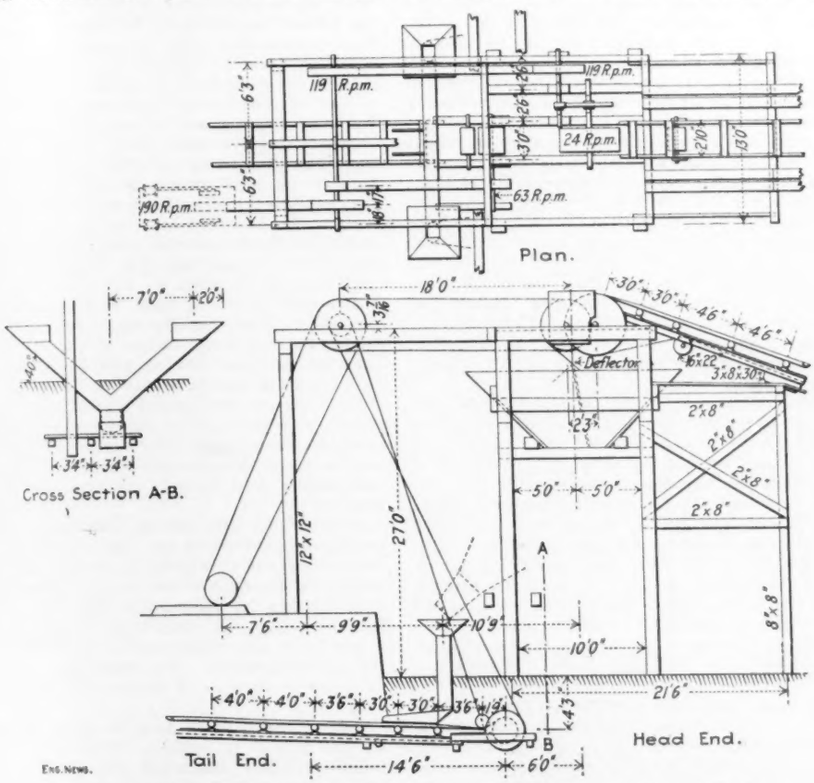


FIG. 2. SKETCH SHOWING ARRANGEMENT OF MACHINERY IN MIXER HOUSE.

THE STRENGTH OF BLACK DIAMONDS was determined in tests made by Prof. A. N. Mitinsky, of St. Petersburg, Russia, a short time ago, according to the Bulletin of the American Institute of Mining Engineers. The method of test used comprised placing the diamond to be tested between metal plates, applying the pressure to these seating plates, and after the test noting the size of the depression made by the diamond in either of the two plates. In one case the two plates were rolled (mild?) steel and soft bronze respectively; in another, soft steel and hard tool steel; in another, mild steel and mild steel. The impressions in the plates were photographed to enlarged scale, and their areas could then be accurately planimetered. The results in the several tests were: 54.3 kg. per sq. mm. without breaking; 80.6 kg. per sq. mm., fractured; 77.5 kg. per sq. mm., fractured; 56.7 kg. per sq. mm., fractured; 70 kg. per sq. mm., fractured. The lowest breaking strength among these, 56.7 kg. per sq. mm., corresponds approximately to 80,000 lbs. per sq. in. The tests indicate that the pressures hitherto considered limiting safe pressures on diamond core drills are needlessly low, and that firmly-set diamonds may be crowded to any desired extent in core-drilling without risk of crushing.

SHOP HINTS FOR STRUCTURAL DRAFTSMEN.*

By John C. Moses,† M. Am. Soc. C. E.

III. Shipping and Erection.

SHIPPING.—The maximum size of a single piece that is to be shipped by rail is fixed by the regulations of the roads over which it has to go. Anything not over 8 ft. wide and 40 ft. long can be loaded flat on a single car. Pieces up to 9 ft. wide can be loaded on edge on practically all roads, but if wider pieces are wanted the clearance limits of the roads over which they are to pass must be examined. When pieces are longer than a car, they must rest on bolsters about 6 ins. high, which, of course, use up just that much of the available head room. These limits are based on cars of standard height, and should not be exceeded without investigation.

Roof trusses that it is desired to ship riveted up complete may be so near the available limit that some projecting gusset, purlin lug or splice will cause them to exceed it. They must then be sent in pieces, or else the details must be modified to keep the truss within the proper size. The draftsman will have to balance the cost of riveting in the field against the cost of the same work in the shop with the perhaps more elaborate details. Field riveting requires more rivets and consequently larger gussets. The rivets will cost a good deal more to drive, say 5 cts. each, for field as compared to 2 cts. each for shop. And if any errors are found when assembling in the field they are much more expensive to correct than they would have been if found in the shop. These considerations lead to the general rule to avoid all field riveting possible.

But there is another point to consider in many cases. Railroads charge for at least 30,000 lbs. of freight per car, and if the actual load is less than this there is a corresponding increase in cost per pound for freight. A job composed of 25,000 or 30,000 lbs. of columns or roof trusses that are 60 ft. long will then cost but half as much for freight if spliced in the field, and this may determine the proper course to pursue. Still smaller jobs, like foot bridges, amounting to three or four thousand pounds, that would require a whole car if their trusses were riveted up in the shop, can be shipped knocked down through the freight office at regular pound rates. If the work has to be teamed from station to site, it will, of course, be easier to handle small pieces.

Long girders are shipped so loaded that they will be pointed the right way when they reach the site. Sometimes, when the proper direction cannot be foretold, it is possible to leave off the shoe plates and thus make the girders themselves reversible. The erector must then rivet on the shoe plates before lowering the girders to place. Projecting gussets or angles on heavy members are liable to injury from having the weight of the member thrown on to them in handling. At least one side of heavy pieces should therefore be free from such projections.

Shipping lists are generally made by the draftsman, and should describe each piece, as well as give its shipping mark. The description should include the principal dimensions, and the items may read as follows:

Two Top Chords, 24 × 36 × 58' 6", Marked U2U4.

Four splice Plates, 16 × ⅜ × 2' 0", Marked BS.

Etc., etc.

Great care must be taken to have all the separate pieces listed. It costs several days' wages to send by express a splice plate that the draftsman failed to list.

An extra allowance of rivets is required to replace those lost or spoiled in heating and driving; 20% is not too many to send additional, as those not used can be returned with the tools. The erector should be furnished an itemized list, showing where the rivets of different lengths are intended to go. If he finds he needs any different lengths, he will then know at once what to send for and will not run out of sizes unexpectedly.

ERECTION.—Erection work is usually done under far less favorable circumstances than the other parts of the manufacture of a bridge. The men are exposed to the weather, and much of the work is done with temporary appliances that would not be considered adequate in the shop. Frequently an old bridge must be replaced by a new one while traffic is maintained, and the work is largely done at night and in a hurry. Sometimes the falsework is in danger from rising streams, and a few hours' delay may mean the loss of a span worth many thousands of dollars. An error that, if found in the shop, could be easily and cheaply remedied, and perhaps would not even be reported to the drafting room, will become a very costly and serious mistake when found in the field. A heavy truss member will be brought out between trains and after several hours' work be gotten nearly into place when it will be found that the draftsman has overlooked a rivet that should have been countersunk, and the member has to be taken out to get at it. The available time has expired and half a night's work has been wasted. Even if the rivet can be cut off without taking out the member, most of the gang may have to loaf while it is being done.

The most effective way, then, to reduce erection costs is to realize the seriousness of errors affecting that part of the work. The same reasons that make mistakes so costly make it worth while to plan carefully for the erectors' convenience in every way possible.

The general drawings of the work are designated as "erection plans," and should be complete enough for all ordinary purposes, the detail plans being kept in the tool-box for occasional reference. The erection plans should give the principal dimensions of the work and show the direction to the nearest important railroad station, street names, points of the compass, or some similar means of fixing the way the structure stands. An index of all the drawings should be given on one of the erection plans. The name of every separate piece that appears on the shipping list should be given in its proper place. Erection plans of truss bridges should give extreme height and width, these dimensions being needed to determine size of traveler. The spac-

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†Engineer of Construction, The Boston Bridge Works, Inc., Boston, Mass.

ing of the floor beams fixes the location of the falsework bents. The clear headroom under portals and bracing and the clear width between trusses should appear; also the distance from masonry to center of end-pin, top of floor beam, or some similar point that will govern the erectors' layout. As a general thing, it is not wise to economize by showing less than the whole length of a bridge span, even when it is symmetrical. For buildings these plans should give grades of all wall and shoe plates, as well as their location from the building lines. Where existing masonry has to be cut away, the plan should show size and location of holes, in order that they may be cut in advance by the mason.

A system of marking that is used with much success by some companies is as follows: The templet maker always writes his "shop mark" and orders on the left end of his templet, this being always the left end, as shown by the detail drawings. The marks are painted on the steel just as they appear on the templets. The name of each member on the erection plan is placed on the end that is to the left on the detail drawing. The result is that the erector has only to place the "marked end" of his piece to correspond with the mark on his drawing. The same system is followed out on all the pieces that make up a complete member, and is of great assistance in the shop. All templets are sent to the shop with the marked ends together, and as a result all punched material reaches the assembling gang with its marks one way, and the riveted members are sent out of the shop, stored in the yard, and finally shipped with a uniformity that is a help in many ways. In order to make this system still more effective, the draftsman should follow regular rules in drawing truss members, columns, etc., laying them out on his detail drawings with the left hand end as the end nearest the abutment, or by some similar rule. This system of marking is the one used on the drawings of the double track plate girder span that appeared as an illustration with Part I. of this article (see Engineering News of March 22, 1906).

The arrangement of details for greatest economy and convenience in erection requires a knowledge of erection methods that is not common. More money can be made or lost in erection than in any other part of a job. The cost of the materials and of the drawing, templet and shop labor, the amounts to add for rent, fuel, office expenses and salaries, and the freight to the site, can all be determined in advance with a small percentage of error. The estimated cost of erection is generally a guess that is liable to vary 25% from the final result. This is due in part to unavoidable circumstances and in part to the fact that erection has been left so largely in the hands of the workmen unassisted by the engineering department. An engineer will see where details can be changed to help erection, and yet not injure the structure theoretically, but a foreman cannot go into that side of the question. The present tendency is to remedy this state of affairs by bringing the erection and engineering departments together for mutual advice and assistance. This, of course, is essential on great enterprises, and will undoubtedly be found profitable on all classes of work. It is hoped that others will add to the following hints from their experience.

ERECTION OF BUILDING WORK.—Pedestals and base plates for columns and wall plates for beams and girders should be separate pieces, and the erection plans should give the grades of their top surfaces. They can then be set in advance to accurate grade and location. Columns should be bolted to the pedestals, as they will then need little guying while the beams are being put in place. All columns should be spliced at one level to avoid interfering with the derricks that rest on the floors and are moved up a column length at a time. The splice should be high enough above the floor to admit of riveting after the floor is laid. The splice plates can be riveted to the lower section, but the rivets nearest the end of the section should be omitted to allow the splice plates to be spread a little. If the vertical distance between the splice plate holes at the joint is made a sixteenth less than the distance between column holes, the sections will be drawn together when pinned up for riveting.

Beams are often connected to columns by resting on brackets, to which they are riveted through their lower flanges. Stiffness is secured by having knees on top of the beams connecting them to the columns. These knees are best shipped loose. They should be drawn with ⅛-in. clearance underneath, as beams nearly always overrun in height on one side, the top and bottom flanges not being square to the webs. When double beams connect to columns or to other beams, parts of their flanges may have to be removed at the ends in order to drive the connecting rivets. Long bolts going through both beams can be used, but are not very satisfactory. If knees are riveted to a beam to receive the web of another beam between them, the drawing should call for a space 1-16-in. greater than the web. A beam having this kind of a connection at both ends is likely to be hard to get in.

Erection plans should show which way the edges of channels turn, the line of the web being dotted in. They should also show the direction of the column webs and the sizes of the beams, as well as their names. All pieces that are identical should have the same mark. If every column has a separate number, the erector will presumably have to overhaul his pile to find the right one when there may be others on top that will do as well. As a matter of fact, he will generally try to find out from his details what ones are alike in order to save himself this trouble. And in that case he is, of course, likely to make mistakes. Tie-rods should be designated by their length, as they are too small for marking by paint.

ERECTION OF ROOF WORK.—Roofs consisting of trusses connected by bracing or purlins can often be planned to avoid any riveting after the trusses are hoisted in place. The parts of the truss are assembled on the ground and riveted together and the truss hoisted as a whole, the bracing and purlin connections being bolted. If rivets are required, a staging must be built at each joint, and the rivets may cost 25 cts. apiece. One man can put in bolts without any staging, but riveting requires a platform to work on and four men in the gang. Common bolts are perfectly good in many places, and it is good engineering to take advantage of their cheapness in such cases. Turned bolts with a close fit in the holes may sometimes be necessary. If they are expected to supplement rivets in the same joints, they must fit the holes as tightly as the rivets. This will require a taper bolt, and the holes reamed for each one as it is put in. They will be very expensive and of doubtful value, and the draftsman should only call for them as a last resort. Places that are too confined to admit of riveting are generally too confined to admit of the proper reaming for and screwing up of bolts.

ERECTION OF PLATE GIRDER BRIDGES.—Plate girder bridges always have the girders shipped whole. In deck bridges the sway frames should be ⅛-in. less in depth than the space they are to go in. In through bridges having floor beams and stringers the floor may be put in place first and the girders moved in sideways, or the girders may first be placed. If the latter plan is followed, the floor beam connection to the girder must be planned to allow the beams a movement along the girder that will separate them far enough to get in the stringers. The stringers have to be put in diagonally past the floor beam flanges and then turned straight. The length of the stringer on its longest diagonal should be figured and the possibility of getting it in fully investigated.

One very common difficulty met with in erection is the drilling of the holes in the stone piers for the anchor bolts. This must be done after the steel is in place. The drills should be at least ¼-in. larger than the bolts to make a hole that will admit the cement around the bolt. But draftsmen call for holes in the shoe plates that are just large enough to admit the bolt and place them under gussets and end sway frames, or through the lower angles of stringers, and seem to think the holes will be marked through on to the stone and the bridge removed while the holes are drilled. Consequently, erectors have to chip out the steel to get their drills in on a slant that will bring the top end where they can strike it. The result is a crooked bolt, or frequently no hole at all. Anchor bolt holes in shoe plates should be ½-in. larger than the bolts and in position admitting of holding the drills vertical, with room to swing a sledge for striking them. Use a washer under the nut to cover the hole.

Another common oversight is to call for field rivets in the bracing over abutments and piers where the stone is so near that the rivets cannot be entered. In fact, the bracing itself is not infrequently found to interfere.

ERECTION OF HIGHWAY BRIDGES.—The erection plan for a highway bridge should fully show the woodwork for the floor and fence. If the bridge is on a skew, the ends of the planks at the abutments must be supported in some way. Similarly, wooden fences and wheel guards have to end somewhere, although draftsmen have a way of leaving such matters undetermined. Roadway plank should be planed on one side to obtain an even thickness, and will then measure less than the nominal thickness. Plank that must break joint should be all of one width. Wooden stringers must be sized to less than their nominal depth at the ends to have them alike, and height of shelf angles must be fixed accordingly. Wooden stringers will shrink, and where they are nearly flush on top with steel floor beams some provision must be made for carrying the planking over the beam without resting on it. Spiking pieces on top of beams are best attached by bolts that go entirely through the wood, for lag screws are much harder to use. All lumber to be painted should be ordered planed, and plans should state the number and sizes of nails required. The woodwork of a highway bridge is often the greater part of the erection, and a large item in the total cost of the job.

ERECTION OF TRUSS BRIDGES.—Truss bridges are erected in two general ways. The floor beams and stringers may first be assembled on the falsework and the trusses bolted up to them as they are erected, the floor beams holding the trusses in place until the top bracing is put in; or the trusses may be put in first and the floor afterwards. The draftsman should be informed in advance of the method that will be used.

Most of the remarks already made will apply to truss spans. In addition, the draftsman must provide suitable clearances for posts entering into chords and like connections. Built members will vary a little from figured dimensions, being slightly out of square or measuring a little larger when several thicknesses are piled up together. Rivet heads will be higher than figured. Most offices have rules for clearances based on their own experience, but if none such are at hand the draftsman may use the following:

Assume all eyebars and plates used singly as 1-16-in. thicker than figured. Where plates are riveted together, assume each one as 1-32-in. thicker than figured. Assume all countersunk chipped rivets as ⅛-in. high, and all flattened or full-head rivets as 1-16-in. higher than figured. Then add ⅛-in. on each side of a member for clearance as it is put into place.

The portals and overhead sway frames are put in last, and should be arranged to go in without spreading the trusses. It is often a good plan to ship them with the top angles separate. These top angles can then be used for temporary bracing; they leave headroom for the derrick cars, which are coming more and more into use for erection purposes.

Pins are driven into place with pilot nuts temporarily screwed on to the ends. Room must be left to get these nuts off when floor beams or bracing connections come opposite the ends of the pins. Room must be left around pin holes for the nut to turn. When the holes come near the edge of an angle, this often requires a filler to be riveted to the web plate of the chord.

Ends of chords and end posts that bear on pins with half holes should be cut to clear each other by ⅛-in. They should not be faced to bear on each other, when a pin is used, for it is then necessary to bolt them rigidly together before boring the pin hole. This makes trouble in the shop, especially when the two members make an angle with each other, as at a hip joint. Pin

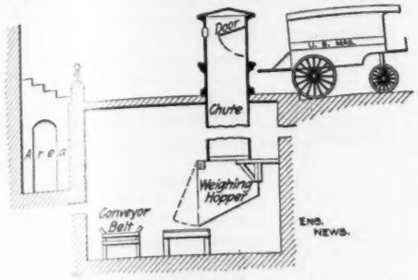


Fig. 1. Mail Delivery System at the Chicago Post Office.

holes are bored with a horizontal boring bar, the members resting on a table or skids.

CONCLUSION.—In the series of articles here concluded the writer has tried to point out to draftsmen the advantage of a knowledge of shop methods and their relative difficulty. The explanations and illustrations given do not exhaust the subject, but will, it is hoped, be of assistance.

THE COST OF 9-IN SEWERS AT GUELPH, ONT.

Some notes on the cost of 9-in. lateral sewers, built (except No. 1, in table) in 1904 as a part of a separate system of sewers at Guelph, Ont., are given in the Proceedings of the Ontario Land Surveyors for 1905, and are republished here in slightly condensed form. The author of the notes was Mr. James Hutchinson, of Guelph. It is to be noted that the sewers were built by day labor, with a single gang of men, mostly Italians, all under one foreman. Ordinary laborers were paid 16½ cts. per hour, but 17½ cts. were given to a few better class men, who handled pipe and sheeting when required. Bricklayers were paid 35 cts. an hour, and the foreman was paid $3.50 a day.

The accompanying table, with its foot notes, shows the character, length and cost of each of 12 laterals:

Cost of 9-in. Lateral Sewers Built in 1904 at Guelph, Ont.

Sewer No.	Length, ft.	Depth, ft. Maximum.	Depth, ft. Minimum.	Depth, ft. Average.	Cost of Excavating, pipe laying & backfilling.	Cost of Labor per foot of sewer.
1	972	12.9	10.4	12.0	$974	$1.00
2	550	12.0	9.3	10.9	421	.76½
3	1,208	12.4	9.5	10.8	911	.70
4	688	12.5	8.9	10.7	478	.69½
5	900	12.7	8.6	10.2	631	.70
6	1,780	13.5	7.0	10.0	1,304	.73
7	1,150	11.2	8.0	9.3	585	.51
8	1,696	12.2	6.5	9.0	930	.55
9	830	10.8	7.6	8.9	465	.56
10	840	9.7	8.0	8.6	474	.56
11	800	9.7	8.0	8.4	351	.44
12	600	9.8	5.2	7.0	235	.39

No. 1.—Begun in 1904; cost increased by difficulties of work after winter set in, and by wet condition of partially dug trenches in spring.

No. 2.—Very bad ground.

No. 3.—Trench yielded many limestone boulders; there was some 100 ft. of quicksand bottom, requiring close sheeting. Otherwise the ground was good, little bracing was required.

No. 4.—Mostly dry gravel; required sheeting.

No. 5.—Chiefly sand, with clay at deepest part. Sheeted for about half its length. Wet bottom; 6-in. subsoil drain beside and a little below sewer.

No. 6.—Middle third of this was most difficult work of 1904. Soil gravel, with water 4 ft. from surface. Diaphragm pumps used ahead, and in the bottom water allowed to flow away in the sewer and subsoil drain.

Nos. 7 and 8.—In adjacent, parallel streets, but there was water-bearing gravel one-third length of No. 8.

Nos. 9 and 10.—In different drainage areas, but both had gravel in deeper portions, sand at upper ends, and subsoil drains; also two manholes in each.

No. 11.—Soil gravel and for the most part dry. Close sheeting not required; occasional braces used. Work was rushed, thus reducing cost.

No. 12.—Ground similar to No. 11.

For further description the paper mentioned may be quoted, as follows:

The cost includes unloading the sewer pipe and the brick from the cars, the building of manholes and the removal of stones and surplus earth from the streets, as well as the excavating, pipe laying and backfilling.

The street surfaces were of gravel. The ground was variable, sand, gravel and clay often being passed through on the same sewer. On the whole, however, the conditions were favorable for economical work.

The width of trench varied from 3 to 4 ft., according to the depth of the sewer and the nature of the ground. Trench screws with 1½-in. gas pipe were used for bracing. For sheeting 1½-in. dry hardwood, 8 ft. long, was found most satisfactory; green lumber will not stand driving. Tamarack, 4 × 6 ins., used for stringers.

The backfilling was put in by hand and was not tamped, but the trenches were flooded with water from the fire hydrants. After the filling had settled for some time, the street surface was made good with the road grader and the steam roller. The cost of this work was not charged to the sewer account.

The manholes were mostly between 300 and 400 ft. apart and were circular in plan, 4 ft. in diameter at the bottom to height of about 4 ft., and then drawn in to about 2½ ft. The labor of building the manholes was between 3 and 4% of the whole cost of the labor.

Reducing the costs given in the schedule to a rate per cubic yard, it is found that the results are fairly uniform and will average between 47 and 50 cts. per cu. yd. of excavation.

To compare these figures with the cost of similar work done by contractor, we must add the cost of materials, the contractor's profit, a percentage for wear and tear and a liberal allowance for unforseen contingencies, liberal because a quicksand bottom or an injury to a workman might easily double the cost of a sewer.

MAIL CONVEYING APPARATUS AT THE NEW CHICAGO POST OFFICE BUILDING.

The new post-office at Chicago has a very complete equipment of special apparatus for the handling and conveying of mail, both in bulk and in small lots, and this plant presents a number of interesting features. Before describing the plant, however, it will be of interest to briefly describe the general situation. The architectural design of the building was apparently made without due regard to its practical use, and when preparations were being made to put it into service it was found that, among other defects, there were no adequate facilities for receiving the great quantities of mail brought by post-office and private wagons. The building, which has been described in our columns, occupies an entire block, but has only one driveway for wagons. This driveway was originally intended to be on the street level, but when it was decided to raise the first floor above this level it became necessary to lower the driveway to the basement level. As the streets at either side limit the length of the inclined approaches, the grades are so steep that at the entrance end, men are stationed to put skids under the wheels of the descending wagons, while at the exit end extra two-horse or four-horse teams have to be hitched to the outgoing loaded wagons, causing some confusion to the street traffic. It is proposed to put in a traveling roadway to carry the wagons up the incline, but there is at present no appropriation available for this.

It was at once evident that the driveway could not be used for all the incoming and outgoing wagons, and the post-office authorities endeavored to persuade the city to vacate the sidewalk on the Dearborn St. side of the building, so that wagons could be driven alongside the building and some arrangement of chutes devised into which the bags and bundles could be thrown. As this is a very busy street in the heart of the business district great inconvenience would have been caused by compelling people to cross and recross the street and the city refused to grant this request. It was then decided to put vertical chutes in the sidewalk on this side of the building, and 14 large rectangular openings were cut in the new 15-ft. concrete sidewalk. One of these is covered by trap doors and has an elevator for the local carriers' pouches brought in by the carriers and the collecting mail wagons. Each of the others is surmounted by a tall iron box about 3 × 5½ ft. and 7 ft. high, set 3 ft. from the edge of the curb, and having an opening in the side facing the street. A similar box (making 14 in all) serves as an office for the man who sees to the unloading. The city has taken steps to compel the removal of these obstructions, but as there seems to be no other way of receiving the mail, it is probable that the boxes will be allowed to remain. The arrangement is inconvenient in that the wagons must back up to the curb, causing interference with traffic, especially as the paving of the street is in a disreputable condition. The throwing of bundles down the shaft into a hopper is also said to result in damage to the bundles and their contents.

With this explanation of the general conditions we may turn to the handling of the mail within the building. Under each of the 13 mail boxes and chutes already mentioned is a 1-ton weighing hopper, in the basement, as shown in Figs. 1 and 2. When a wagon has backed up to one of the boxes, the door is opened from below upon a signal from the driver, which rings a bell and

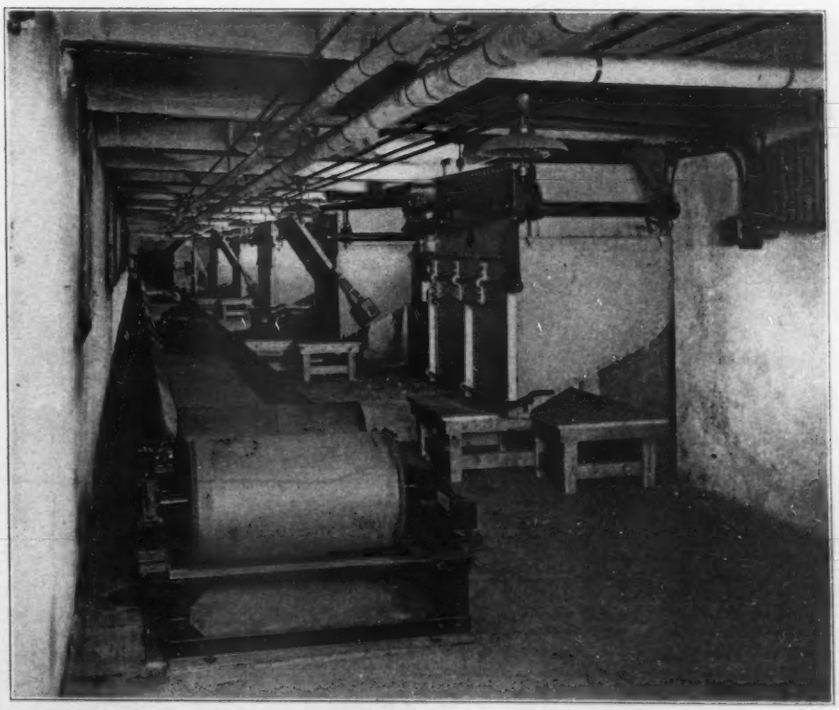
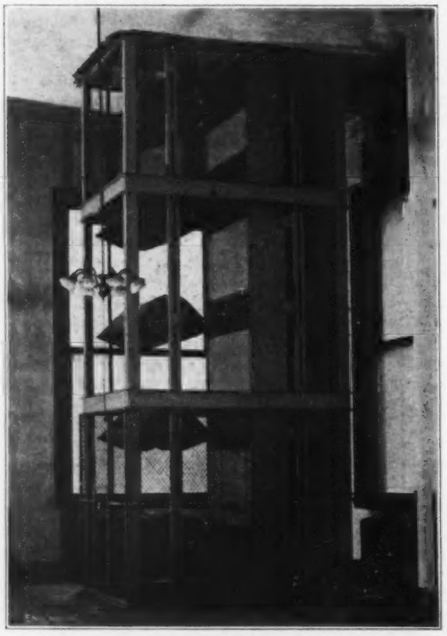
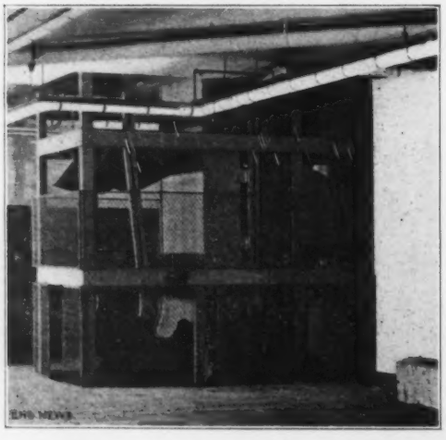


FIG. 2. RECEIVING AND WEIGHING HOPPERS AND 36-IN. BELT CONVEYOR FOR MAIL BAGS: CHICAGO POST OFFICE.
Jeffrey Mfg. Co., Columbus, O.: Builders.

Discharge Section (Second Floor).



Intermediate Section.



Receiving Section (Basement).

Fig. 3. Mail Bag Elevator.

lights an electric lamp. The bags are thrown through the door of the box and fall into the weighing hopper, the weight of the contents being recorded by a recording beam scale. A row of six of these hoppers, with the conveyor serving them, is shown in Fig. 2. When the door of the hopper is raised, the sacks and bundles fall out upon a table, from which they are thrown by hand upon a horizontal belt conveyor 36 ins. wide, running at 75 ft. per minute, and serving the two rows of chutes. This delivers the bags to an inclined belt conveyor running in the same direction at 500 ft. per minute, and this in turn delivers the bags to a 48-in. belt conveyor at right angles to the other and running at 90 ft. per minute. The effect of the high-speed inclined conveyor is to straighten out the bags so that they fall crosswise upon the 48-in. belt, which is the proper position for their delivery to the elevator buckets.

The 48-in. conveyor carries the bags to the boot of a vertical elevator and discharges them automatically into the steel buckets, which are 4 ft. 6 ins. long and 40 ins. wide, attached to two roller chains of 24-in. pitch, running at a speed of 60 ft. per minute. This elevator, Fig. 3, discharges the mail bags, pouches and carriers' satchels upon tables, where they are sorted and loaded upon trucks for delivery to the "State" sorting points of the distributing department. There are three separate conveyor systems, as above described, on the east (Dearborn St.) side of the building, each system consisting of three belt conveyors and an elevator.

The belt conveyors are all flat, with stationary wooden sides to prevent any bags from falling off; this arrangement requires much less vertical space than if concave belts and troughing rollers had been used. The rollers are 4 ins. diameter, about 4 ft. apart, and run in ball bearings. The end or driving rolls are driven by high-speed roller chains, direct-current motors furnishing the motive power. These chains are of the construction shown in Fig. 4. The side bars A, A are of die-forged steel and have the projections B, B, which are finished to a smooth surface, and bored out for the hard steel pin C, which is keyed in place. These side-bars are then case-hardened. The hard-steel roller, D, runs on the projections or hubs, B, B. The smooth and hard finish makes the chains run easily and with little wear or noise. The head sprockets of the elevators are geared directly to their motors.

On the west (Clark St.) side of the building there are four bucket elevators, very similar to the one shown in Fig. 3; the steel buckets are 4 ft. 6 ins. long and 29 ins. wide, spaced 4 ft. apart, and attached to two lines of roller chain of 2 ft. pitch. The elevators are loaded by hand from the receiving platform at the driveway in the basement. Two of them discharge on the first floor and two on the second floor.

The cars of the narrow-gage tunnel system of the Illinois Tunnel Co., already described in our columns, will bring mail from the railway stations, and also a large part of the newspaper and commercial mail from newspaper offices and large establishments. For handling this a separate conveyor system is installed on the west side of the building. As shown in Fig. 5, the tunnel runs under the depressed driveway in the basement, and is of enlarged section, with two tracks, and a 30-in. belt conveyor running between them for a distance of 140 ft. This conveyor is horizontal (and below the level of the rails) for 90 ft. and then rises on an incline into a chamber under the driveway of the building, where it discharges its load automatically upon another inclined belt conveyor (at right angles to it). This second conveyor is 5 ft. wide and discharges the mail at the receiving floor in the basement. These belts run at 400 ft. and 250 ft. per minute respectively. In order to provide for the great amount of mail which is expected to be delivered at the post-office by the tunnel cars, the conveyors are made of sufficient capacity to discharge 300 bags per minute during the busy hours of the day. At the same point in the tunnel the cars are loaded with outgoing mail, the sacks being sent down chutes to a platform between the tracks.

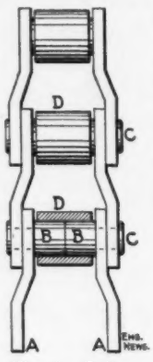


Fig. 4. Roller Driving Chain for Belt Conveyors.

A separate conveyor system running from the first to the second floor handles all the loose mail, packages and pouches delivered by the public through the letter drops and windows of the city division. This system consists of two inclined belt conveyors 36 ins. wide, having steel pans riveted to the belts, as shown in Fig. 6. The men at the receiving tables inside the letter drops and windows throw the letters, etc., into this conveyor, which is directly behind them and which discharges the mail onto tables on the second floor, where it is sorted and distributed to the proper subdivision for the several States. These belts run at a speed of 60 ft. per minute.

Still another conveyor system is provided in the money order division to carry the money order applications from the clerks' windows to the cashier's desk, a distance of 71 ft. This is shown in Fig. 7, and is of interest in that it has to carry light slips of paper which have practically no inertia. This conveyor consists of a 6-in. horizontal rubber belt having longitudinal ribs to prevent the sheets of paper from sticking to it. The belt is enclosed by a wooden casing, so that the pieces of paper cannot fall or be blown off, but there has been some little trouble from static electricity in the belt causing the

FIG. 6. CONVEYOR FOR LOOSE MAIL FROM DROP BOXES AT THE CHICAGO POST OFFICE.

papers to rise and strike the bottom of the counter. The belt runs at a speed of about 120 ft. per minute, and is driven by a motor and machinery concealed beneath the counter on which the money orders are made out.

The power used is as follows, but the motors are of considerably higher capacity: For the receiving system on the east side of the building, 2 HP. for the first belt, 1 HP. for the second belt and 4 HP. for the elevator, making 7 HP. for each of the three groups, or 21 HP. in all. Of the four elevators on the west side, receiving mail from the driveway and tunnel, two require 3 HP. and two 4 HP. each. The two inclined belt conveyors for city mail require 4 HP. each, the money order conveyor requires 1 HP., and 35 HP. is required to drive the two conveyors carrying mail from the tunnel into the basement of the building. The total power requirement is therefore about 100 HP.

The conveyor equipment was built and installed by the Jeffrey Mfg. Co., of Columbus, O., and we are indebted to that company for drawings, photographs and other information. For drawings of the tunnel connections we are indebted to Mr. George W. Jackson, Chief Engineer of the Illinois Telephone Construction Co.

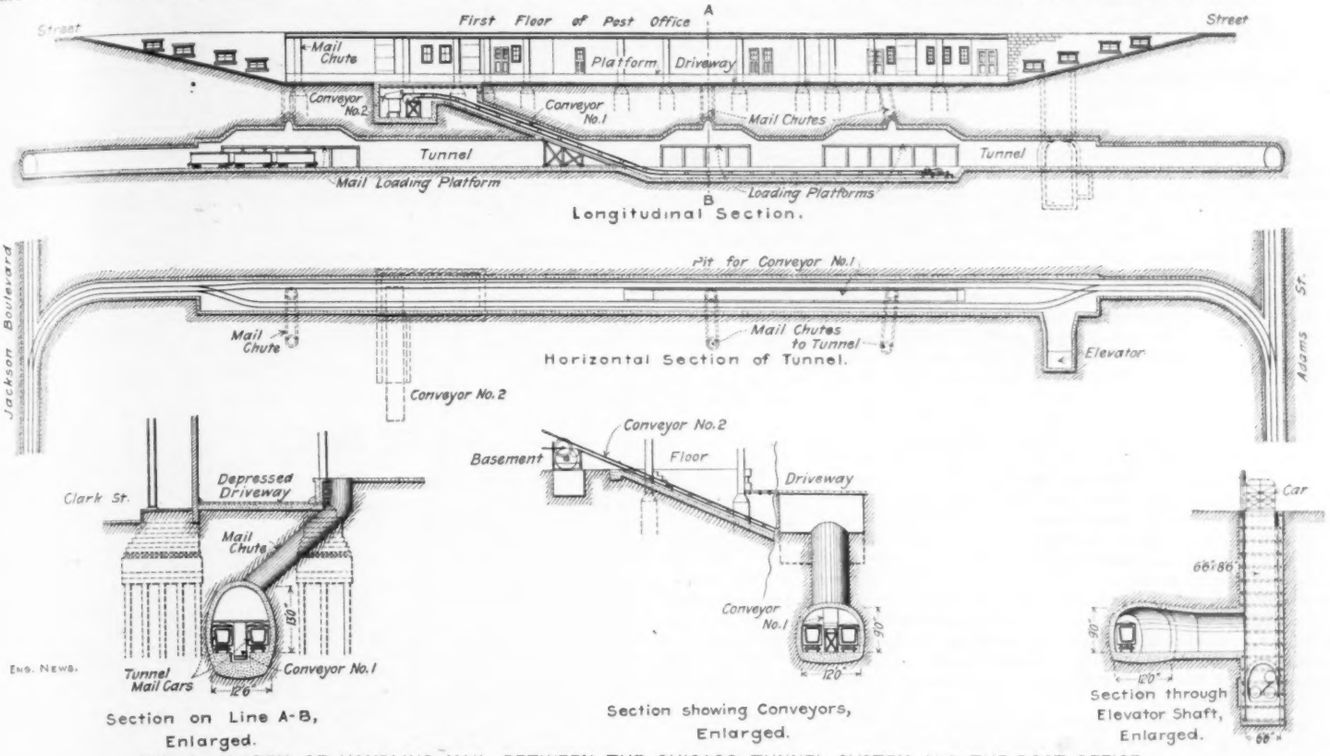


FIG. 5. SYSTEM OF HANDLING MAIL BETWEEN THE CHICAGO TUNNEL SYSTEM AND THE POST OFFICE.

A REMARKABLE PUMPING ENGINE TEST.

A high test performance was made by a 20,000,000-gal. pumping engine built for the Bissell's Point pumping station of the water-works of the City of St. Louis, Mo., by the Allis-Chalmers Co., of Milwaukee, Wis. The acceptance tests showed a duty of 181,068,605 ft.-lbs. per 1,000 lbs. steam, or 158,851,000 ft.-lbs. per million heat units. This is among the very highest records of pumping engine performance. A table of duty records was given in Engineering News of Sept. 27, 1900 (p. 214), which shows that a 6,000,000-gal. Nordberg pumping engine at Pittsburg, Pa., gave a duty of 162,900,000 ft.-lbs. per million heat units; records of 155,200,000, 158,070,000, and 156,800,000 also appear in this table. The Bissell's Point engine thus is a close second to the Nordberg engine.

Another noteworthy feature of the test was the large bonus earned by the builders on the showing of the test results. The specifications required a duty of 135,000,000 ft.-lbs. per 1,000 lbs. steam, and placed a bonus of $1,000 per million foot-pounds on a performance better than specified. The test results thus indicate a bonus earned of something over $46,000.

The following gives the particulars of the test, for which we are indebted to Mr. Wm. A. Hoffman, Mechanical Engineer in charge. The test was run under the direction of Mr. B. C. Adkins, Water Commissioner, and Messrs. M. L. Holman and J. A. Laird, Consulting Engineers; by Mr. Hoffman acting for the city and Mr. T. T. Hubbard acting for the Allis-Chalmers Co., assisted by employees of the Water Department as observers.

The engine tested is the last of a series of three installed at the above station, and is of the vertical triple-expansion type, with three single-acting outside-packed plungers, each located directly under one of the cylinders. The bed-plates rest on solid rock foundations; the main pillow blocks and the cylinders are supported by columns of the A-frame pattern. The cylinders are 34, 62 and 94 ins. in diameter by 6 ft. 0 ins. stroke; the water plungers are 33⅞ ins. in diameter, by 6 ft. 0 ins. stroke.

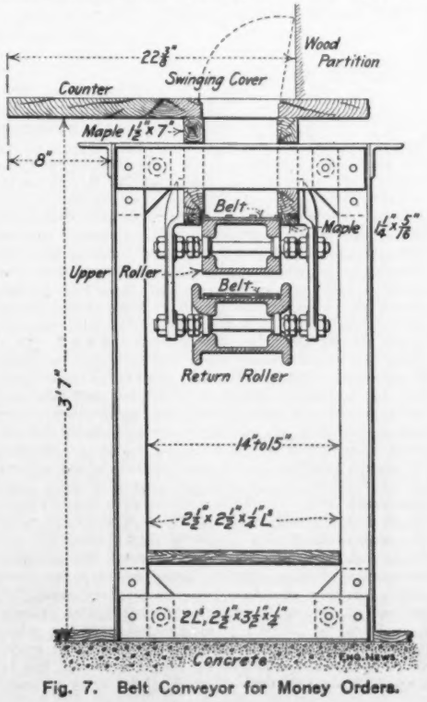


Fig. 7. Belt Conveyor for Money Orders.

Before the official test the plungers were carefully calibrated by micrometer calipers checked by steel tape measurement of circumferences. The strokes of all plungers were also carefully measured. The pump valves were inspected and found to be tight under full pressure. All instruments, gages and scales used on the test were carefully tested and calibrated. The pumpage was computed as plunger displacement, allowing for plunger leakage, which was weighed and found to be 16.77 gals. per hour. The head on the discharge main was read by means of a mercury column, the scale on which read in feet head. The suction head was indicated by a float gage.

The specifications and contract required that

> In order to determine the amount of steam used by the engine, the water will be weighed twice; that is, the feed water going into the boiler and the condensed steam coming out of the engine.

Accordingly, the condensation from the condenser, jackets, receivers and drips from stuffing boxes was weighed as received from the engine and delivered in the boiler room, and was found to check the feed-water by 0.12%. This being a reasonable check the water as weighed in the engine room was taken as the steam used. Steam at 140 lbs. pressure was furnished, containing 0.13% moisture.

The pumpage was 20,070,690 gals. in the 24-hr. period, against a head of 100.021 lbs. per sq. in. at the discharge pipe (the contract requiring 100 lbs. pressure). The indicated horse-power was 865.23, and the steam consumption was 10.60 lbs. per I. HP.-hr., as shown more fully in the following table of results:

RESULTS OF DUTY TEST.
Date, February 20 and 21, 1906.

Duration of test	24 hours.
Diameter of cylinders	34, 62 and 94 ins.
Stroke of engine	72 ins.
Diameter of plungers	33⅞ ins.
Average steam pressure at engine	140.24 lbs. per sq. in.
Average pressure in first receiver	26.36 " "
Average pressure in second receiver	2.77 " "
Average vacuum pressure by cards	13.21 " "
Average barometer pressure	14.46 " "
Average net head pumped against	238.2323 ft.
Average revolutions per minute	16.539
Piston speed per minute	198.44 ft.

Total water pumped	20,070,690 gals.
Total water received from engine	220,129 lbs.
Average moisture in steam	0.13%
Indicated horse-power	865.23 HP.
Delivered horse-power	842.69 HP.
Per cent. friction	2.60%
Average moist steam per I. HP. per hour	10.60 lbs.
Average dry steam per I. HP. per hour	10.59 "
Average B. T. U. per I. HP. per minute	201.39 B. T. U.
Mechanical efficiency	97.40%
Duty per 1,000 lbs. of steam	181,098,605 ft. lbs.
Duty per 1,000,000 B. T. U.	158,851,000 " "
Thermal efficiency	21.06%

PLUMBING A LEANING RETAINING WALL AND BRIDGE ABUTMENT.

By Lindsay Duncan.*

A retaining wall abutment 36 ft. high and 80 ft. long, which had tilted forward to an alarming degree because of settling of the foundations, was recently plumbed up and made stable by the writer by a method which may interest engineers. The wall referred to formed the south abutment of the new Main St. Viaduct now under construction at Pueblo, Colo. It was built of gravel concrete and was designed as a gravity wall with a top width of 2½ ft. and a bottom width of 7½ ft., with a footing course 8 ft. wide and 4 ft. thick. As will be seen from the dimensions given the width at the base was only 9-10 of the height, instead of 4-10, according to the usual practice. Moreover, the foundations had been carried down to only 7 ft. below the surface and rested on a bed of adobe clay streaked with gravel.

The settlement and tilting was due to the softening of the adobe by ground water. In settling, the wall tilted forward, as shown by the view Fig. 1. This prevented the erection of the viaduct and, of course, prohibited any backing being placed behind the wall. In consequence, the writer was called upon to devise means of restoring the wall to a vertical position, and of increasing the stability by such an amount as would enable it properly to fulfill the functions of a retaining wall.

The methods adopted in restoring the wall are illustrated by the drawings of Fig. 2. The face of the wall was first shored up by 8 × 8-in. timbers as a safeguard against further tipping. Trenches 4 ft. wide and 6 ft. deep were then dug along both faces. In each of these trenches there was sunk a row of Raymond concrete piles, spaced 2½ ft. on centers, along the front face, and 5 ft. on centers along the rear face. These piles were driven to a depth of about 20 ft. below the trench bottoms, and their tops were finished off at a level about 2 ft. below the bottom of the abutment footing. When the piling was completed, work was begun cautiously undermining the wall by taking out one, at a time, 6-ft. strips of earth between trenches and filling each excavation immediately with 1-3-6 concrete in which were embedded 1-in. corrugated bars. The succession of concrete beams thus built formed a continuous footing 4 ft. thick and 16 ft. wide carried on concrete piles reaching well below the adobe stratum into the gravel of the old Arkansas River bed.

The only trouble experienced in the construction of the footing came from ground water. Pumping was necessary throughout the construction, and all openings had to be sheet-piled. The method of settling the original wall onto the new footing was to wash out the thin wedge-shaped layer of intervening earth by means of water jets, the movement being assisted by jacks on the shoring timbers. When the wall had thus been made to take its proper upright position, the interstice between the old and new footings was filled with grout.

A solid foundation for the wall having been obtained in the manner described, the next move was to reduce its instability due to its narrowness of base. The two footing courses, with a combined thickness of 6 ft., were extended backwards to form a lip, as shown by the drawings of Fig. 2. In addition, the rear face of the wall was relieved of tension by bars spaced 2 ft. apart and connected at top and bottom as shown. These bars being in place, the rear face of the wall was given a 3-in. coat of concrete to protect them from rust. Against the front face of the wall a series of buttresses or counterforts were built. There were 12-in. slabs of reinforced concrete taking support on the new footing and bearing against the wall face, to which they are bonded by steel rods, as shown.

The work described was done by the Bullen Bridge Co., of Pueblo, Colo., according to plans prepared by the writer.

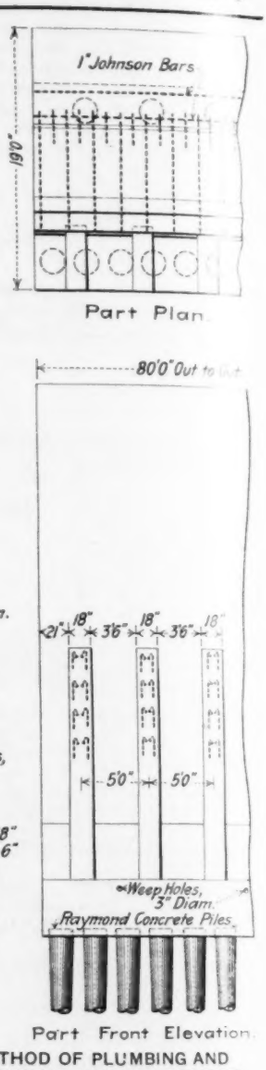


FIG. 2. SKETCHES SHOWING METHOD OF PLUMBING AND STRENGTHENING A LEANING RETAINING WALL ABUTMENT.

*Consulting Engineer, 512 Boston Building, Denver, Colo.

Fig. 1. View of Abutment Retaining Wall Out of Plumb, Due to Defective Foundations.

AN INVESTIGATION OF STREET PAVEMENTS at Binghamton, N. Y., has been made and reported on by the Municipal Committee of the local Chamber of Commerce. The detailed study was made by Mr. Byres H. Gitchell, Secretary of the Chamber. Mr. Gitchell's conclusions, which have been endorsed by the Committee and by the Board of Directors of the Chamber, may be summarized as follows: (1) Where assessments for benefits prevail, abutting property owners should be permitted to select the kind of pavement to be laid; (2) the present city specifications for street work should be revised by experts; (3) if more asphalt is laid, the city should employ an expert to supervise the work; (4) a closer inspection of street work, and more thorough tests of materials used therefor, should be had; (5) street openings in pavements should be restricted, and all pipe connections should be made and curbing set before new pavement is laid; (6) all new pavements should be put under construction as early in the spring as possible; (7) asphalt patching, where large portions of the surface are worn, results in excessive cost, and in its place complete resurfacing, with such material as is deemed best, should be substituted, "but in any case, brick, stone or creo-resinate wood block should be used between the car tracks and immediately outside the rails; (8) complete resurfacing of worn asphalt, under expert supervision, with a ten-year guarantee, can be done for about "$1 per sq. yd., which is considerably less initial expense than would be involved in resurfacing with any other material under a similar guarantee." The report states that Binghamton has 253,990 sq. yds. of pavement, laid at a total cost of $564,740. Of this, 117,291 sq. yds., costing $310,984, are of asphalt, laid between the years 1889 and 1898 under a five-year guarantee. The remainder, or 136,699 sq. yds., are of brick and cost $253,756. With one exception, all the brick has been laid since 1897 under a ten-year guarantee. The report from which the foregoing information has been taken has been printed as a 34-page pamphlet, in which the local situation is reviewed in considerable detail and in which there are also given several tables of statistics on the mileage and cost of asphalt, brick and macadam pavements in 97 cities as collected by the Committee.

WATER ANALYSIS IN RELATION TO TYPHOID EPIDEMICS was recently discussed at some length by Dr. Ernest C. Levy, Director of the Laboratory of the City Water Department of Richmond, Va. The paper is well worth the attention of those responsible for public water supplies, particularly in that it exposes some fallacies held by the general public and many practicing physicians regarding the possibility of detecting the typhoid bacillus in water suspected of being the cause of a typhoid epidemic. As the paper may be obtained, we presume, by sending a stamp to the author, we need give here only the conclusions:

(1) No satisfactory method is known for the detection of B. typhi in water.

(2) Were such a method known, it would be of limited application on account of the fact that when an outbreak of typhoid fever leads to an investigation, the water supposed to have been infected is no longer available for examination.

(3) Although the direct detection of B. typhi in water is impossible, yet the importance of bacteriological and chemical examination of the suspected water must not be under-estimated, but it should be combined with a thorough sanitary survey of the field, and a competent study of the phases of the epidemic. This will almost invariably solve the question whether the outbreak was caused by drinking-water.

(4) This method of attacking the problem has the further advantage of throwing light on the sanitary quality of the water in question, apart from the special outbreak of typhoid fever, and, moreover, if it is found that the water is not responsible, or responsible only in part, full information will be gained along many lines, thereby suggesting the steps to be taken for future protection.

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The burning of the Fall River Line steamer "Plymouth" while lying at her pier in Newport harbor last week adds another to the long list of steamer conflagrations which go to prove the tinder-box character of all our American steamboat construction. There was a watchman on board the "Plymouth" and two watchmen on the dock when the fire broke out, but the flames spread so fast that of the 19 employees on board one was burned to death, two had to jump overboard to escape the flames, and several others had narrow escapes. It is beyond question that if the fire had started when the steamer was in regular service, making her night run between Fall River and New York, the great majority of her load of sleeping passengers would have been burned to death.

Again and again has Engineering News condemned the tinder-box construction of American river and sound steamers. At the time of the "General Slocum" holocaust, in New York harbor 18 months ago, we explicitly stated that the loss of nearly a thousand lives was not because the "Slocum" was an old boat, not because she was overloaded, not because of neglect of captain, crew or government inspectors, but because she was built of such inflammable construction as to directly invite such a disaster as occurred. And we further said that the "Slocum" was no worse a fire risk, in all probability, than the so-called finest examples of the steamboat builders' art plying on the Sound or the Hudson River. This was proved (if proof were needed) by the burning some years ago of a palatial Fall River Line steamer while lying at her wharf in Newport, and the burning of the "Plymouth" under almost identical circumstances affords still another illustration of the astounding rapidity with which these highly decorated floating tinder-boxes can burn when fire once gets a foothold.

Not once, but repeatedly, has this journal held up to condemnation the present construction of steamboat superstructures. Not once, but repeatedly, have we urged the adoption of fire-resistant construction on shipboard. And we intend to lose no opportunity to record each new object lesson of this class until a reform in construction is brought about that will render impossible a repetition of the "Slocum" horror.

We do not know whether the managers of the Fall River Line will proceed to erect another tinder-box on the steel hull of the "Plymouth" (which is said to be little injured), but one would suppose that with the burning of the "Slocum" still fresh in the public mind, and the rapid spread of the flames on the "Plymouth," as an object lesson, there might be a possibility that a departure in construction might be adopted right here and now. Does not the great and wealthy corporation which owns the Fall River Line owe this much to the traveling public whose lives are at risk?

In the course of the examination of engineers before the Senate Committee on the Panama Canal, a point on which the opponents of the lock canal have been laying much stress is the location of the Gatun locks. Mr. Lindon W. Bates on March 10 first brought the matter forward, claiming that the designers of the lock canal had made a huge blunder by locating a three-lock flight in the Gatun hill, whereas, according to Mr. Bates, there is not room enough to locate three locks there with 900 ft. usable length except by having the ends of the locks located on bad foundations in the alluvial valley. (See map of the Gatun dam and locks in our issue of Feb. 22, p. 206.)

This testimony has been controverted by Messrs. Stearns, Noble and others who have since appeared before the Committee; but all possible doubts on the matter were dispelled on March 20 by the receipt of the following cablegram from Chief Engineer Stevens:

Replying to Secretary Taft cable 18th I say limits in length required for twin triple locks, Gatun, 3,100 ft.; in width, 300 ft. Have fully developed by 100 borings for 3800 ft. in length and 300 ft. width on exact site, perfect foundations, and can build two sets of three locks, each 1,150 ft. long if necessary, and perhaps much longer. Conditions ideal for construction and permanency.
Stevens.

During Gen. Peter C. Hains' testimony on March 27, a letter was presented written by Chief Engineer Stevens, dated Dec. 19, from which the following quotation was read:

Of the engineers who are now on the Isthmus in the employ of the Commision, of all ranks, of all degrees of experience and knowledge, I have yet to find a single man who is in favor of a sea-level canal. Most of them are very outspoken against such a proposition; and while it may be said that they are not men of world-wide reputation in technical knowledge and experience, I claim that an intimate knowledge of the conditions obtained by a residence of months and years on the ground is of far more value than any theories or conclusions which may be drawn from existing works in other parts of the world which bear not the slightest resemblance to the proposition at Panama.

It will be seen from the date of the above letter that it was written before the report of the Board of Consulting Engineers was presented, and hence the opinions of the engineers working on the Isthmus must have been formed from their own knowledge of the conditions and uninfluenced by the conclusions of the Board, which were made public two months later.

The city council of Chicago is in dense sanitary darkness, judging from its action on the house drain controversy. Instead of prohibiting the use of tile pipe for house drains (that portion of the sewer connection between the foot of the soil pipe and a point just outside the house wall), as has been done for years in all well-regulated cities, it has gone so far as to refuse to impose any effective test on the tightness of tile house drains. For nearly a year the health commissioner, chief sanitary inspector, the daily press and many others interested in the health and prosperity of Chicago, realizing that the prohibition of tile house drains was impossible, have been supporting an ordinance providing that such drains be subjected to a 10-ft. water pressure test. Finally, by a vote of 43 to 15, the city council has substituted a 2-ft. for a 10-ft. head in the test, and has vested the inspection and tests of these drains in the public works instead of the health department. Doubtless some members of the Chicago city council are honest in their belief that the interests of owners and renters of small houses demand the permissive use of tile pipe, but that in the long run this will prove a false economy all engineers and sanitarians know full well. Assuming that tile pipe is to be permitted for house drains, as it should not be, only mercenary reasons can be urged against submitting it to a rigid test when it is put in. In addition, the regulations should require that it be always open to ready inspection. House drains, like all other plumbing and sanitary construction within buildings, should be inspected by the health department during construction, at completion and frequently thereafter. A 2-ft. head test of house drains, honestly carried out, is a little, but only a little, better than a mere visual inspection. The people of Chicago will suffer, both in pocket and health, for the sanitary ignorance, or worse, of its city council in its refusal to insist on strong, tight house drains.

Occasional derailments which have occurred in the New York subway have so far caused no serious results, but they have led to a discussion as to the possibility that some future derailment at high speed, perhaps on a curve, may throw a car against the columns with force enough to cripple several of them and bring down a section of the roof of the tunnel. It can hardly be denied that such an occurrence is a possibility; yet the actual chances of its occurring, so far as can be seen, are very small. The columns are so close to each other that a derailed car at high speed is much more likely to slide along against the row of columns than it is to catch one of them and force it out of place. Besides this, even if a row of a dozen columns were to be knocked out of place, it is by no means certain that the roof would fall. On the four-track section the beams overhead would probably still be sufficient to carry the load. It does undeniably seem a little risky, even to an engineer, to run trains at 50 to 60 miles an hour past a row of columns, when one recalls the numerous accidents on steam railways where derailed trains have struck buildings, water towers, or other structures and caused serious casualties; but the saving fact in the subway is that the columns are close together.

It would be a simple and not enormously expensive matter, of course, to rivet a guard rail or channel to the columns on each side at proper height for a derailed car thrown against the columns to slide along it; or a low concrete wall might be made by filling in between the columns. Unless some serious accident of the sort above outlined should occur, however, there seems little likelihood that any such construction will be undertaken.

There appears to be fair prospect that Congress will at this session enact the long-advocated law removing internal revenue taxation from alcohol used for mechanical or industrial purposes, provided it is treated by a so-called "denaturizing" process, which renders it unfit for drinking. In Germany and Great Britain, for many years, denaturized alcohol has been untaxed, and an enormous consumption of alcohol in various manufacturing processes has resulted. Alcohol lamps are used for illumination with a Welsbach mantle, which give 1,471 candle-power hours per gallon burned as compared with 783 candle-power hours from a gallon of kerosene burned in a kerosene lamp. Tests at the University of Brussels have shown that alcohol at 31 cts. per gallon is as cheap for lighting purposes as kerosene at 15 cts. per gallon.

Alcohol is also extensively used abroad in small motors instead of gasoline. From 5,500 to 6,000 alcohol engines are said to be in use in Germany. The efficiency is substantially the same as that of gasoline, about 1 pint per horse-power hour. While the heat units in the alcohol are less than in gasoline, alcohol vapor can be compressed to a much higher degree than gasoline without causing premature explosion and the thermal efficiency of the alcohol engine can be made about 30% as compared with 21% for the gasoline engine. The exhaust from the alcohol engine contains less injurious gases than the exhaust from gasoline motors, and it has been proposed, therefore, to build alcohol locomotives for use in mines.

It is believed that alcohol can be produced in the United States from many waste products of farms, plantations, etc., at a cost of not more than 25 cts. per gallon. In Cuba it is said to be produced on sugar plantations from refuse molasses at a cost as low as 10 cts. per gallon.

MUNICIPAL SEWAGE PURIFICATION EXPERIMENTS AT COLUMBUS, O.

The sewage purification experiments at Columbus, O., to a review of which a considerable amount of space has been devoted in our last and present issues, will receive the careful attention of engineers interested in sewage treatment. As stated in our previous issue, they are notable both in themselves and as being the first extensive municipal experiments in this country on the treatment of sewage, made prior to embarking on the installation of sewage works. Experimental work has been done by several other municipalities in connection with existing plants, notably at Worcester, Mass. Some valuable studies of various phases of sewage treatment in connection with existing works have also been made at Brockton, Mass., and Pawtucket, R. I.; in addition, as everyone interested in the subject knows, the workings of the various sewage purification plants in Massachusetts have been carefully studied and reported on by the Massachusetts State Board of Health for many years past, and the board just named has operated the Lawrence Experiment Station for nearly twenty years. At Columbus it was wisely decided to expend the equivalent of practically a year's interest charges on the cost of the proposed works in studying the various methods most promising for local use before beginning the preparation of detailed plans. This was all the more desirable because it seemed probable that Columbus would wish to make use of methods or combinations of methods not previously tried in this country on a large scale. The authorities at Columbus and the engineers engaged by them, we understand, feel that the expense of the experimental work has been more than justified by the results obtained and that the experience secured will save the city many times its cost in construction and operation of the permanent works.

It is interesting to note the attitude of the specialists engaged on the Columbus problem towards the relation of the sewage of that city to public water supplies drawn from the Ohio River. The effluent from the proposed sewage works will discharge into a river joining the Ohio 100 miles below, and this junction point is about 100 miles from Cincinnati, O., and Covington and Newport, Ky. No public water supplies are drawn from the tributary of the Ohio, and practically none from the Ohio itself, until the three cities just named are reached. All these cities take their water supply from the Ohio. Cincinnati is building large water purification works. In view of these circumstances and others which it is not necessary to enumerate here, the chief problem of sewage disposal at Columbus was held to be one of "eliminating nuisances in the immediate vicinity of the city." This conclusion is interesting, particularly as it was reached before the recent decision in the Chicago-St. Louis drainage canal case. The distance between Chicago and St. Louis, it will be remembered, is some 350 miles by way of the canal, the Des Plaines, Illinois and Mississippi rivers. The safe distance between a source of public water supply on a given stream and the outlet of a sewerage system is always an interesting subject for speculation; all the more so when the sewage is partially purified before being discharged into the stream. In the Columbus case probably very few engineers will question the decision, already noted, to the virtual effect that public water supplies need not be taken into account.

The schedule used at Columbus for collecting samples of sewage for analysis was commendable. The taking of half-hourly samples ensured a thoroughly representative sewage, and the omission of sampling on every eighth day, thus moving the day omitted forward each week, prevented any abnormal conditions due to the repeated omission of the same day in the week. This feature, combined with the apparent continuous operation of the plant, instead of shutting it down on Sundays, makes the results more than usually comparable with those obtained in actual practice.

The Columbus practice of screening all the sewage as the initial step in its treatment is in line with the almost universal foreign sewage works practice. Such screening will probably be more generally adopted in this country hereafter, and where already practiced it is likely that more thorough screening will, in many instances, be substituted in the future.

The tank at Columbus used as a grit chamber seems to us to have been of unusual proportions. The report contains no explanation of why these were adopted. The original so-called grit chamber was of the identical size and shape of the sedimentation and septic tanks, namely, 8 ft. wide, 40 ft. long and about 8 ft. deep. It being found that this tank did not serve the real purpose of a grit chamber, namely, the deposition of heavy mineral matters or grit only, and the forward passage for further subsidence of practically all the lighter organic matters, the chamber was reconstructed to a size 5 × 39½ ft. in plan and 3 ft. in depth. The original grit chamber gave a passage period of 1.3 hours, and the remodeled of 0.29-hour, as compared with 8 and 6 hours, respectively, for the two sedimentation tanks. Practically all of the grit chambers heretofore used, as we understand the practice, have afforded far shorter periods of detention than even the remodeled grit chamber at Columbus, the distance of flow across these tanks being generally a few feet only.

The conclusion that septic tanks of a capacity equal to eight hours' sewage flow are sufficiently large seems rational. In the early history of the septic tank, this journal raised the question as to whether a holding capacity equal to 24 to 36 hours' flow was not unnecessary, and also prohibitive in cost. It was held as probable that if the tank could be so designed that the sludge deposit was not carried forward by the current a tank capacity sufficient to afford the optimum period of sedimentation was all that need be provided. The tendency in America has been to provide small septic tank capacities compared with those abroad, but many of the septic tanks built in Great Britain by others than the Septic Tank Syndicate or the engineers connected therewith have been of less holding capacity than was originally proposed by Mr. Cameron.

Perhaps the most unique suggestion in the Columbus report is that the disposal of the sludge from the septic tank might without objection be by dilution when the river is in flood. The city has available, however, a large amount of land at the site of the disposal works which can be used for sludge disposal by burial. Viewing the Columbus sewage works as designed wholly for the elimination of a nuisance in the immediate vicinity of the sewage outfall, we see no reason why sludge disposal by dilution at Columbus should not be adopted. If, however, there is any possibility of the sewage influence extending 200 miles to the water supply intakes of Cincinnati, Covington and Newport, then the discharge of large accumulations of sludge at intervals might be considered highly objectionable, notwithstanding the suggestions in the report to the effect that the chances for the survival of pathogenic germs in the septic tank sludge are small. In the present state of sewage disposal in relation to water supplies, we can but conclude that the dangers from sludge disposal by dilution at Columbus will be very small indeed compared with those resulting from the almost universal sewage disposal and water supply practice of this country.

As between contact beds and percolating filters, the Columbus experiments seem to be more conclusive as regards the latter. This is proper enough, inasmuch as the probabilities were from the start that percolating filters rather than contact beds would be eventually employed. But inasmuch as the valuable report on the Columbus experiments will be widely drawn upon for information regarding all the methods of sewage treatment tried, it is only fair to state that the contact beds at Columbus seem to have been operated at a considerable disadvantage as compared with usual practice. Such beds abroad almost invariably receive effluent from septic tanks, whereas at Columbus the contact beds were dosed, for the most part, with sewage which had been settled only, and much of that for very brief periods. It should be noted, however, that the report indicates that in the opinion of the observers at Columbus it mattered less than has been commonly supposed whether septic or settled sewage was applied to the contact beds.

The main lesson to be learned from the contact beds at Columbus seems to be that of the four parts of the contact-bed-cycle—filling, standing full or contact, draining and resting—all but the latter, or resting, should be as brief as practicable and the resting should be relatively long. This seems to accord with generally accepted ideas, so far as filling and emptying is concerned, but more stress is usually laid upon the importance of contact. Possibly more preliminary treatment than was given, as a rule, in the Columbus experiments, and better underdrainage of the beds, would have lessened the necessary resting period. It should be remembered, however, that there is a very strong tendency in Great Britain to adopt for new works percolating filters rather than contact beds, and that this appears to be largely due to a high rate of clogging and loss of voids in the contact beds.

If the contact beds at Columbus were at a disadvantage compared with those elsewhere, so, it seems to us, were the percolating filters, in that the latter were perhaps of too fine-grained material and evidently not thoroughly underdrained. In view of the wide use of revolving sprinklers abroad, it would have been interesting to have seen a sprinkler of this type tested at Columbus. We do not feel at all sure, however, that such a test would have done more than prove the impracticability of a movable sprinkler in very cold weather. Abroad, the revolving sprinklers are much more used than the stationary, but there seems to be a tendency to employ the stationary ones more generally at present, and, so far as we know, they have given satisfaction where used. A well-designed nozzle certainly seems to possess decided advantages as compared with the necessarily small perforations where perforated pipes, whether moving or fixed, are used.

As to intermittent sand filters, it may be said that while the experiments seemed to prove conclusively that, local conditions considered, sand filters are not desirable at Columbus, yet valuable suggestions for their use elsewhere may be drawn from the report. These suggestions would be applicable where either good sand was available at low prices or where, for any reason, a high degree of purification was considered necessary, in which latter case either the well-established Massachusetts practice of intermittent sand filtration at low rates might be followed, with whatever preliminary treatment was made desirable by local conditions, or else sand filters might be operated at high rates as a finishing process. The Columbus experiments, it will be remembered, indicated that the effluent from percolating filters might be sand-filtered at the high rates of 2,000,000 to 3,000,000 gals. per acre daily.

Inasmuch as the Columbus experiments and report are pioneers and are bound to have a considerable influence upon the nomenclature of the newer processes of sewage treatment in this country, it seems unfortunate that the term "sprinkling filters" was adopted. "Intermittent sand filters" and "contact beds" or "filters" are quite satisfactory as descriptive terms of the processes involved, or if not satisfactory are as nearly so as can be hoped from one or two words. "Sprinkling filters," by contrast, merely indicates the method of applying the sewage, and gives no hint of the operation of the bed itself. We think, as has been previously stated in these columns, that "percolating filters" is a more satisfactory term than "sprinkling filters," and hope to see it even more generally used than it already is.

The Columbus experiments, as a whole, emphasize anew the importance of dividing the work of sewage treatment into two distinct parts—the preparatory and the finishing processes. The experiments for the first time, so far as we remember, give prominence, or even mention, to colloidal matter as an element in the design and

operation of American sewage disposal works. As might be expected, the septic tank is not overestimated as an adjunct of sewage works, and, as already indicated, we are pleased to note that the proposed period of detention in a septic tank is relatively low. Engineers will await with interest the results obtained from a large installation of percolating filters operated at so high a net rate as 2,000,000 gals. per acre per day. Remembering that a non-putrescible effluent is the object at Columbus, that besides filtration the sewage is first to be screened, then passed through septic tanks, and, finally, after filtration, to have two hours of sedimentation, the proposed rate seems practicable. Equally high rates under no more favorable conditions are reported from many places abroad, and the results and recommendations as a whole are in general accord with British tests and practice, unless it be in the short passage-period through the septic tanks. As will be obvious to all competent to judge, the tests and report constitute a distinct addition to the science and art of sewage treatment. Other communities, however, should not conclude that they can in every case design sewage purification works in the light of the information made available by the experiments at Columbus and elsewhere. Sewage is so complex and local conditions vary so widely that experimental work will yet be required in many other communities before a sewage disposal plant can be designed out of hand, as can a bridge. Variations in the character of sewage, the filtering material available and other local conditions will in many other of our larger communities make it desirable that they, as did Columbus, should devote the sum equivalent to pay a year's interest on the cost of necessary work to experiments for the sake of obtaining information to guide them in the design of their works. The scores upon scores of abandoned and reconstructed sewage works in Great Britain, antedating the period of scientific investigations there, show what may be expected from adopting some new-fledged and untried scheme of sewage treatment absolutely without knowledge of its adaptability to local conditions. The capital thus sunk abroad, combined with the unnecessarily high operating expenses of unsuccessful works, would in every instance have paid many times over the cost of experimental investigations. It is only fair to say, however, that until recently engineers, chemists and bacteriologists were not so well equipped as now to carry out such investigations as have been made at Manchester, Birmingham, Leeds, London, and many other places in England, and as have just been brought to such an eminently happy conclusion at Columbus.

LETTERS TO THE EDITOR

A Caisson Tunnel Invention 19 Years Old.

Sir: As a contribution to the literature on the subject of "Who Killed Cock Robin?" or rather as to who it was in New York that first suggested the use of pneumatic caissons for tunnel construction, I enclose plan and sections for a proposed Inter-Metropolitan railway, which was published by a friend of mine in the year 1887.

His idea was to provide two tunnels across the East River for a circular railway, which would permit the running of the elevated trains in New York and Brooklyn across the river between the two cities, the river tunnels to be constructed as shown in the sections by pneumatic process.

Very truly yours, Veritas.

New York, March 28, 1906.

[The illustrations sent by our correspondent show a caisson cross-section for a double-track railway tunnel strikingly similar in its general form to the caissons used in the Seine Railway tunnel shown in our issue of Feb. 15.—Ed.]

Another Way to Start a Transit Stuck on the Tripod.

Sir: The suggestion of "S. D. C." on "How to remove an instrument stuck on its tripod," published in your issue of March 15, reminds me of another method, which I have used, to accomplish the same result.

Raise one leg of the tripod as high as it will go and place the point of it against the side of a building with the other two legs in a vertical position. By moving the bottoms of the two vertical legs slightly from the wall the horizontal leg will act as a lever and cause a small distortion in the tripod head, which will loosen it so that it can be easily unscrewed.

I have used this method successfully on transits that were badly stuck.

Yours truly, J. W. Sussex.

Pathfinder, Wyo., March 23, 1906.

Guessing at What Becomes of Water Supplied by the Battle Creek and Other Water-Works.

Sir: It is a fact well known among city water-works men that a large part of the water pumped never reaches the consumers. Plants having nearly all their services metered will ordinarily measure from 40 to 70%, only. The remainder escapes in various ways, some of which can be determined with a fair degree of accuracy, and others with a great deal less. Some sources of loss may be called mathematically indeterminate, as they vary largely from time to time, and any attempt to estimate them is usually little better than a wild guess.

The writer has seen tables giving the total amount of water pumped, taken from pumping station records, and accounting for it all by giving the amounts used and wasted in the different ways. Such a table is usually made up in a more or less haphazard manner by distributing the total amount of unmetered water among the different items according to the best judgment of the author, and is often quite misleading.

The table published in your issue of March 15 (p. 307), giving statistics of the plant at Battle Creek, Mich., shows evidence of having been made up in that way. The different items differ widely from the writer's experience, and from the averages given in some of the later water-works books. A single illustration will suffice. Estimating six people to a tap, 52,324 gallons daily were used for extinguishing fires. This would allow a good-sized stream to play for 20 minutes each year on every building in the city which takes water. No wonder our insurance rates are being raised!

Yours truly, Edward H. Cowan,
Supt. Marion City Water Co.

Marion, O., March 19, 1906.

[It is only fair to Mr. Brigden, Superintendent of Water-Works at Battle Creek, to say that in a note following the table in question Mr. Brigden disclaimed that his figures were close ones. He stated that he hoped to make them closer in the future; that in his opinion few detailed figures as close as his had ever been made public, and appealed to others to publish like figures. On referring to the original report, we find that the average daily water consumption at Battle Creek in 1905 is given as 1,190,000 gals., so the estimate of 4½% for fire purposes would give approximately 54,000 gals. a day used for extinguishing fires. The total pumpage for the year was 434,000,000 gals., 4½% of which is about 20,000,000. This would be equivalent to the combined capacity of the two 2,000,000-gal. pumps, forming the Battle Creek pumping equipment, for four days, or one pump alone for eight days.

Certainly, Mr. Brigden is in better position than anyone else to estimate how much water was actually thrown on fires during the year, but unless there were some heavy conflagrations it seems unlikely that so much water was used for fire protection. It must be remembered that the figure given is for a specific year, and not an average for a series of years, and so, as already intimated, might possibly have been high in 1905. Now that the figures are up for discussion, we may add that an allowance of only 3% for slip in "two Blake compound, duplex, condensing steam pumps," each 2,000,000 gals. daily capacity, seems more open to criticism than a 4½% allowance for water used at fires.

The original table and all the foregoing comments upon it illustrate anew the unfortunate conditions under which most of the water-works of the country are operated. Mr. Brigden's report shows that the cost of water at Battle Creek in 1905, including pumping and other operating expenses and only 4½% capital charges on the total cost of the works, was $72 per 1,000,000 gals., or $85.68 per day, or $32,248 a year. And yet, with 88% of the services metered, he can definitely account for only 53% of the pumpage. In other words, he can only "guesstimate," as an engineer has put it, how $15,000 worth of water is distributed between various public and private uses, leaks, defective pumps and meters that under-register. And he can do far better than can most water-works superintendents.—Ed.]

Foundations of Reinforced Concrete Arch Bridges.

Sir: For a number of years the writer has followed with interest the descriptions of concrete arch bridges in the technical journals and has often wondered why he never met with a word of criticism or protest, either editorially or from correspondents, against the evident disregard of the importance of the question of the foundations of these structures in most of the illustrations and texts. Year after year such bridges are described with hardly a suggestion as to the character of the material in the beds of the creeks or rivers on which the piers and abutments rest and with insufficient information regarding the depths of the foundations, the assumed unit pressure on the supporting strata in the bed of the river, etc. And yet these points are fully as important as, for instance, the manner in which the steel rods, imbedded in the mass of the concrete, are located and bent and twisted; sometimes to the utter bewilderment of the uninitiated. Furthermore, these foundations, as far as can be judged by the scanty information furnished, are in many instances open to criticism, and it is on this point that I beg space to enlarge somewhat.

The drawings of the Charley Creek Bridge, Wabash, Ind., published in your issue of March 15, seem to illustrate well the points which we wish to bring out. The following statements will pass unchallenged:

(1) The foundations of a bridge are, at the very least, of equal importance with any other part of the same.

(2) In view of their weight and the disastrous consequences of unequal settling, the designing of proper foundations for concrete arch bridges becomes of greater importance than for bridges with steel superstructures.

(3) The laws of nature which apply to the foundations of a reinforced concrete arch bridge are identical with those according to which other bridge foundations have to be planned, and are not affected or modified by the ingeniousness which the inventor of a new patent steel concrete arch system applies to the designing of the superstructure.

Taking now as an illustration the above mentioned Charley Creek Bridge and assuming, which is plainly shown (though not mentioned) in Fig. 1, that the pier

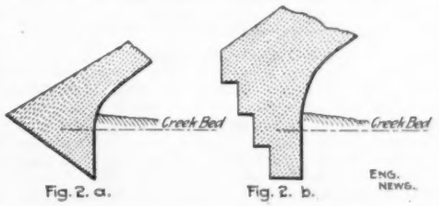


Fig. 2. a. Fig. 2. b.

and abutments are not founded on rock, we find the center pier carried down to 3 ft. below creek bed, with a width of the footing of 8 ft.; the abutments, on the side towards the creek, carried down also to a depth of 3 ft., by a width of about 4 ft., then a step up of about 18 ins. to another bearing of 4 ft. width. We may state that the foundations of this bridge are typical of those of numerous arch bridges for highway traffic all over the country. Comparing such foundations with those of highway bridges with steel or wooden superstructures: No competent engineer will design the stone work of a bridge, in cases where no rock is encountered, without carrying the foundations down to, at least, 4 or 5 ft. below the bed of the creek or river, and, in cases where scouring is to be anticipated, to greater depth. He will also try to keep the pressure on the supporting strata in the river bed within an amount warranted by experience, say, ordinarily, 3 to 4 tons per square foot. By scaling the number of square feet in the longitudinal section (Fig. 1) of the Charley Creek Bridge and estimating the weight of concrete at 140 lbs. per cu. ft., of earth fill at 90 lbs. and macadam floor at 120 lbs., we obtained a pressure of approximately 6 tons per sq. ft. of bearing of the middle pier, not including any moving load.

The saving of material in the abutments by means of steps is also not the practice in the building of highway bridge abutments, because it is, justly, deemed desirable to have a uniform, level bearing of the footing courses on the supporting stratum in the river bed. The ingenuity displayed in this respect by the builders of reinforced concrete arch bridges is astonishing: The designs range from the sharp pointed, slanting foundation shown in Fig. 2a through all possible variations, to the abutment with numerous steps of Fig. 2b.

If we now consider that our knowledge is in no branch of engineering more defective than in regard to the resistance of strata, other than rock, in the beds of creeks or rivers to superimposed loads, and that it is therefore the more necessary to be on the safe side, to treat every doubt as a direct, proved menace to the safety of the structure, the short-sightedness of this policy of cutting down the dimensions of the abutments is apparent.

The concrete arch, properly designed, is a type of bridge superior to the steel bridge with stone abutments. There is no reason why it should not last as long as any work built by man, and the failures which have occurred

are not to be attributed to any inherent weakness or fault of the system. It is a strange fact that the failures of the reinforced concrete arch bridges at Paterson, N. J., which were due to faulty foundation, have not brought a more general appreciation of the importance of intelligent and conscientious designs of the foundations of such bridges.

Carl Gaylor, C. E.

900 Wainwright Bldg., St. Louis, Mo., March 21, 1906.

Filing Methods for Engineers in Private Practice.

Sir: In your issue of Jan. 4, 1906, E. B. H., of Seattle, requests information regarding the most up-to-date method of filing such office maps and plans as accumulate in the practice of a civil engineer. This query was answered in your issue of March 8, 1906, by Messrs. W. D. Sell and Ernest McCullough. Mr. Sell's method certainly can lay claim to being original, to say the least. I, however, cannot see the propriety of overloading drawings with such an aggregation of numerals and letters. Mr. McCullough devotes a large space to this subject, containing much relevant and irrelevant matter.

I think many of the difficulties experienced in filing maps could be avoided, if sheets were used instead of long rolls. Mr. F. Lavis, in his paper on "Methods of Location on the Choctaw, Oklahoma & Gulf Railroad" (Trans. Am. Soc. C. E., Vol. 54, June, 1905, p. 128), brings up the question of sheets or roll maps. It is to be regretted that the discussion on this part of the paper was so small.

	CASE	DRW.	NO.	SCALE	DATE
DRAWING					
TRACING					
PRINT					
NEGATIVE					
DESCRIPTION					
NOTE BOOK NO.	PAGE NO.		COMPUTATION BOOK NO.		PAGE NO.

Index Card for Drawings.

My individual opinion is that all original drawings should be made on sheets, using a number of standard sizes. Those advocated by Mr. McCullough are very convenient: 24 × 36, 18 × 24, 12 × 18 ins. For smaller than this last size I think 8½ × 10½ ins. should be used, as this is the standard letter size. It is often convenient to attach a blue print to a letter, and for this reason they should be of the same size for ready filing. It is becoming the practice to make specifications 8½ × 10½ ins. in size. The U. S. Reclamation Service now publishes the contract drawings with the specifications, using the same width as the depth of the specifications, viz., 10½ ins. Every drawing for reduction should have the scale drawn on.

Flat maps take up less room than rolled maps. Where it is desirable to show long stretches, tracings can be made of convenient lengths.

For the purpose of indexing, a ruled book with printed headings should be used, in which all drawings should be recorded. The accompanying table is suggested as a suitable form.

The drawings can be placed in cases and drawers in two ways. The drawers can be of such siz as to hold 20 drawings, and would be numbered on the outside like this: 1—20, 21—40, 41—60, 61—80, 81—100, etc., the drawing being placed in numerical order. Or else the drawers can be labelled according to subjects, as culverts, bridges, water tanks, etc.

Until the drawings reached a large number, it might be best to place them in numerical order, regardless of subjects, and later on arrange them in order of subjects. For this reason in the record book it would be well to enter the No. of case and No. of drawer in lead pencil, to be entered later in ink as soon as a permanent arrangement was established. Tracings should be kept in pasteboard tubes placed in a framework of pigeon-holes.

In addition to the book record of drawings there will be needed a card index, so as to readily find a desired drawing. I enclose herewith a specimen record card which I think is very nicely arranged for the purpose of recording all needed data. [The card is photo-reproduced herewith. The original was 5 × 3 ins.—Ed.]. The only objection to the card is that the top space should be a trifle wider.

Yours truly,

Emile Low, M. Am. Soc. C. E.

153 Lancaster Avenue, Buffalo, N. Y., March 10, 1906.

Notes on the Stress-Deformation Curve in Concrete Beams.

Sir: The extensometer tests on large sized concrete beams made at the University of Illinois, under the direction of Prof. A. N. Talbot (Eng. News, Aug. 11, 1904), and at the University of Wisconsin, under the direction of Prof. F. E. Turneaure (Eng. News, Sept. 8, 1904), show that the flexure theories in common use for concrete beams do not apply over the wide range of these tests. The extensometer tests on concrete in compression and in tension made at Columbia University under the direction of Prof. I. H. Woolson (Eng. News, June 1, 1905), explain in a large measure the reason, which must be sought in the assumption of the stress-deformation curve.

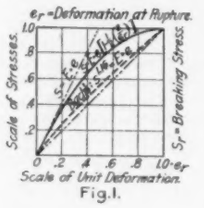


Fig. 1.

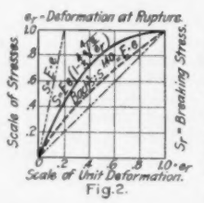


Fig. 2.

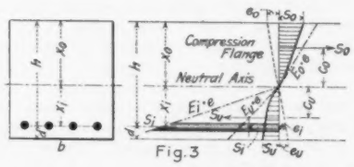
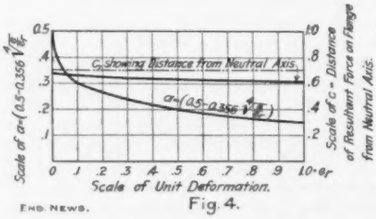


Fig. 3


c, Showing Distance from Neutral Axis.
Scale of Unit Deformation.
Scale of c = Distance of Resultant Force on Flange from Neutral Axis.
ENG. NEWS.
Fig. 4.

A reasonable expression for this curve would be:

$$s = E\,e\,(1 - E'\,e^n),$$

in which e denotes the unit deformation corresponding to the unit stress s, and E, E' and n coefficients determined from experiments. At the point of rupture this curve should be nearly horizontal, hence:

$$E' = \frac{1}{(1+n)\,e_r^n}, \quad s = E\,e\left[1 - \frac{1}{1+n}\left(\frac{e}{e_r}\right)^n\right],$$

$$\text{and} \quad s_r = E_r\,e\,\frac{n}{1+n},$$

in which e_r = unit deformation corresponding to unit stress s_r at rupture.

For $n = \infty$, or $E' = 0$, we get the rectilinear diagram, that is, $s = E\,e$.

For $n = 1$ (Fig. 1), we get the parabolic diagram, with $s_r = \frac{1}{2}\,E\,e_r$. Both of these curves are used in existing theories of concrete.

For $n = \frac{1}{4}$ (Fig. 2) we get:

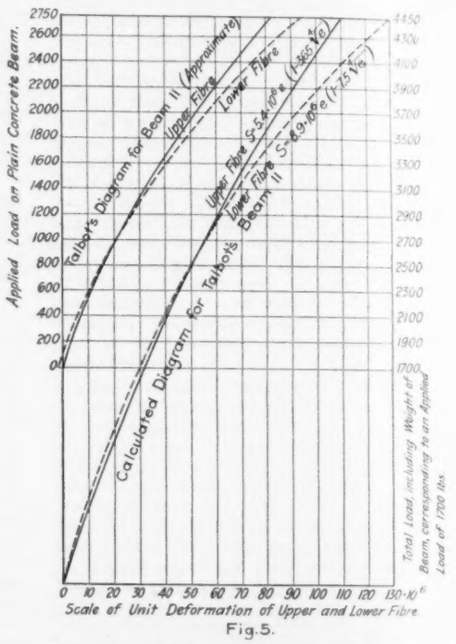
$$s = E\,e\left(1 - \frac{4}{5}\sqrt[4]{\frac{e}{e_r}}\right),$$

$$s_r = \frac{1}{5}\,E\,e_r$$

For concrete in compression this curve corresponds closely with an average of Woolson's diagrams for samples 3,022, 3,023 and 3,026, composed of 1 cement, 2 sand and 4 broken lime rock, about two months old; also with Talbot's beams, composed of 1 cement, 3 sand and 6 broken lime rock, about two months old; and fairly well with Turneaure's beams, composed of 1 cement, 2 sand and 4 broken rock, hardened in water and three months old. The average curve for Prof. Woolson's above named samples corresponds closely enough with:

$$s = 6.25 \cdot 10^3\,e\,(1 - 4\sqrt[4]{e})$$

Prof. Woolson's diagrams for concrete in tension are too irregular for definite conclusions; but they do not seem to preclude the possibility of the diagram for tension being of a form similar to that for compression, and it will be of interest to see what such assumption leads to, as regards plain concrete beams. With extensometer measurements beginning at zero stresses


Applied Load on Plain Concrete Beam.
Talbot's Diagram for Beam 11 (Approximate)
Upper Fibre
Lower Fibre
Calculated Diagram for Talbot's
Beam 11
Upper Fibre $S = 5.54 \cdot 10^6\,e\,(1 - 1.365\sqrt[4]{e})$
Lower Fibre $S = 8.9 \cdot 10^6\,e\,(1 - 1.35\sqrt[4]{e})$
Total Load, including Weight of Beam, corresponding to an Applied Load of 1700 lbs.
Scale of Unit Deformation of Upper and Lower Fibre.
Fig. 5.

there would be no difficulty in finding the stress-deformation curve. Prof. Talbot's diagrams appear to ignore the weight of the heavy beams, which must correspond to a scale-load of about 1,700 lbs. The lack of accurate data as to the relations between concrete in tension and in compression makes it difficult to determine the deformations caused by said weight.

For a rectangular reinforced concrete beam let h = height of beam, measured from center of metal to compression face; b = width of beam; f = area of reinforcement; and M = bending moment. With reference to Fig. 3 it will be assumed that:

$$s = E\,e\,(1 - E'\sqrt[4]{e}\,e), \text{ and } \frac{e}{x} = \frac{e_o}{x_o} = \frac{e_i}{x_i} = \frac{e_u}{x_u}$$

The resultant force on the concrete flange is found to be:

$$S = b\int_0^x s\,dx = b\,E\,e\,x\,(0.5 - 0.445\,E'\sqrt[4]{e})$$

$$= b\,E\,e\,x\left(0.5 - 0.356\sqrt[4]{\frac{e}{e_r}}\right) = b\,E\,e\,x\,a,$$

and the distance of same from the neutral axis:

$$c = \frac{b}{S}\int_0^x s\,x\,dx = \frac{x}{2}\left(1 - \frac{1}{27 - 24\,E'\sqrt[4]{e}}\right)$$

$$= 0.692\,x\left(1 - \frac{1}{27 - 19.2\sqrt[4]{\frac{e}{e_r}}}\right)$$

The values of a and c are represented graphically in Fig. 4. The expression a E e represents the mean stress on the flange.

VIRGINIA AND KENTUCKY RAILROAD

OFFICE OF THE CHIEF ENGINEER (Name of City, etc.)

RECORD OF DRAWINGS

No.	Date.	Subject.	Size of Sheet. Inches.	Scale.	No. of Case.	No. of Drawer	
1	July 1, 1900	Culvert Sta. 600	24 x 36				
2	" 2, "	Bridge No. 1410	"				
3	" 3, "	Water tank, Castlewood	"				
4	" 10, "	Turntables, Scranton	"				
	11, "	Station, Summerset	"				
5-1	Aug. 1, "	" Louisville	"				
5-2	" 1, "	" "	"				
5-3	" 1, "	" "	"				
5-4	" 1, "	" "	"				

The general expression for the location of the neutral axis is

$$\frac{x_1}{x} = \sqrt{\frac{E_s\left(1-\frac{d}{x_1}\right)^2(0.5-0.445\,E'_s\sqrt[4]{e_s})+E_s\frac{f}{b\,x_1}}{E_s(0.5-0.445\,E_c'\sqrt[4]{e_c})}}$$

which, however, becomes much simpler when the tensional value of the concrete is ignored and also for plain concrete beams. The value $\frac{x_2}{x_1}$ for any e_0 can be found by a couple of approximations after first assuming an approximate value of x_1 and e_0. With this value found the stresses on the section are determined and we have

$$M = S_s(x_1 + c_s) + S_c(c_c + c_s) = S_s(x_1 + c_s) - S_c(x_1 - c_c).$$

Fig. 5 shows to the right the curves representing the deformation of the upper and the lower fiber of a plain concrete beam, corresponding to Prof. Talbot's beam No. 11, as calculated under above assumptions. Under a total load of 4,450 lbs., the assumed point of rupture, the calculated stress in tension would be 231 lbs. and in compression 372 lbs. Prof. Talbot's diagram is represented to the left; the agreement of these curves, as far as they extend, is so close that there appears to be no material error in the assumption. It seems permissible to use the coefficients thus found and shown on Fig. 5 to approximate the initial deformations, from which Prof. Talbot's measurement and diagrams for the reinforced beams start, in order to find the stress deformation curve. For that stage of the beams in which cracks appear on the tension side and the tensional value of the concrete should be ignored, the curve for the compression (an average for five beams) was found to be:

$$s = 3.77 \cdot 10^6\, e\,(1 - 3.67\sqrt[4]{e})$$

By comparing this with the coefficient found for the plain beam it appears that a more accurate value of n would have been $^3/_5$.

By ignoring the weight of Prof. Turneaure's beams and the initial stresses on the concrete arising from swelling in water, the average curve for six beams was found in the same manner to be:

$$s = 6.45 \cdot 10^6\, e\,(1 - 3.68\sqrt[4]{e})$$

Only the total deformations have been considered, as the stresses in beams under maximum loads depend on same. The elastic deformation curve as well as the permanent set, is of equal interest and importance in other respects.

Prof. Bach's exponential form of curve has been indicated by dotted lines in Figs. 1 and 2. While correct enough within their limits, they cannot apply too great stresses and should be used with caution.

The conclusions drawn from a few published diagrams, drawn on a small scale, are of interest and practical value only in so far as they serve to produce more correct results in working up records of experiments.

N. Werenskiold.

Dallas, Tex., Feb. 6, 1906.

Concerning the Survey of the Springfield Mine.

Sir: I was very much interested in the last part of the abstract of the paper by Mr. M. F. Peltier as it appeared in your issue of March 8, 1906. The first reading made evident that a line N. 72° 57′ E. must be "swung to the right" 17° 22′ instead of 16° 44′ to become the line S. 89° 41′ E. Checking the traverses for the two surveys, I find that the line underground, however, is not N. 72° 57′ E., but that it is N. 73° 20′ E. Hence the underground line A B must be "swung to the right" 16° 59′ and not 16° 44′, nor 17° 22′. The data and computation for the underground survey stands as follows:

Courses.	Assumed bearings.	Distances.	Latitudes. N.	Latitudes. S.	Departures. E.	Departures. W.
A to 1	West	125.53	....	.....	....	125.53
1 to 2	N. 88° 45′ W	166.60	3.63	.....	....	166.56
2 to 3	S. 2° 15′ W.	42.00	....	41.97	....	1.65
3 to B	S. 0° 18′ E.	49.50	....	49.50	0.26	.....
			3.63	91.47	0.26	293.74
				3.63		0.26
				87.84		293.48

$$\frac{293.48}{87.84} = 3.341 = \tan 73°\ 20'; \quad \frac{293.48}{\text{Sin } 73°\ 20'} = 306.34.$$

The bearing of the closing line B A then is N. 73° 20′ E., and the length is 306.34 ft., a discrepancy of 0.61 ft. from the length of the line as computed from the surface traverse; or an error of 1 in 500, if we accept one or the other as correct. In certain kinds of work this would be intolerable. The circumstances present at the time of the survey would generally control the method to be adopted in the adjustment of this error. In most instances it would perhaps be as easy to make a resurvey. The equality of the polygon is disturbed and the error of the closing line is more than doubled when the lines are "swung to the right" according to Mr. Peltier's "corrected bearings," as follows: the line B A 17° 22′; the lines A 1, 1 2, and 2 3 16° 44′, and the line 3 B 18° 20′.

The "corrected bearings" above referred to give the following traverse:

Courses.	Bearings.	Distances.	Latitudes. N.	Latitudes. S.	Departures. E.	Departures. W.
A to 1	N. 73° 16′ W.	125.53	36.14	.....	....	120.22
1 to 2	N. 72° 1′ W.	166.60	51.44	.....	....	158.46
2 to 3	S. 18° 59′ W.	42.00	.....	39.71	....	13.66
3 to B	S. 18° 2′ W.	49.50	.....	47.07	....	15.32
			87.58	86.78		307.66
			86.78			
			0.80			

This gives the bearing of the line B A as S. 89° 51′ E. instead of S. 89° 41′ E.; and the length 1.32 more than the survey underground indicated. Disregarding the discrepancy in the length of the line, the corrected "corrected bearings" and the traverse of the underground survey stand as follows:

Courses.	Bearings.	Distances.	Latitudes. N.	Latitudes. S.	Departures. E.	Departures. W.
A to 1	N. 73° 1′ W.	125.53	36.67	.....	....	120.05
1 to 2	N. 71° 46′ W.	166.60	52.13	.....	....	158.23
2 to 3	S. 19° 14′ W.	42.00	.....	39.66	....	13.84
3 to B	S. 16° 41′ W.	49.50	.....	47.42	....	14.21
			88.80	87.08		306.33
			87.08			
			1.72			

This, as expected, gives the bearing S. 89° 41′ E. and the length 306.34.

Yours truly,

John Berg.

Iowa State College, Ames, Iowa, March 14, 1906.

[Proof of the above was submitted to Mr. Peltier, whose reply is printed below.—Ed.].

Sir: I note what Mr. Berg says in regard to the traverse survey of the Springfield mine, and wish to state that the original traverse notes, calculated over one year ago, were mislaid, so the strange part about traversing the new ones is that the underground line checked with the surface line when there was an error in the traverse, but the reason it did is that the course from 3 to B should read S. 0° 18′ W. instead of S. 0° 18′ E.

Herewith are the corrected notes of underground survey which checks within reasonable limits considering the obstructions that were encountered through the main air course.

UNDERGROUND SURVEY.

Course.	Bearing.	Distance.	N.	S.	E.	W.
A to 1	West	125.53	....	.....	..	125.53
1 to 2	N. 88° 45′ W.	166.60	3.63	.....	..	166.56
2 to 3	S. 2° 15′ W.	42.00	....	41.96	..	1.64
3 to B	S. 0° 18′ W.	49.50	....	49.49	..	0.26
			3.63	91.45		293.99
				3.63		
				87.82		

The bearing of the closing line of this survey or the line A B is found as follows:

$$\frac{293.99}{87.82} = 3.347 = \tan 73°\ 22'.$$

The bearing of the line A B underground is therefore N. 73° 22′ E. To find the length of this line we have $\frac{293.99}{\sin 73°\ 22'} = 306.84$. The length of the line A B underground is therefore found to check the length of the same line on the surface within reasonable limits considering the condition of the survey, but the underground bearing is N. 73° 22′ E. and the surface line is S. 89° 41′ E., so the underground survey must be swung 16° 57′ to the right.

The corrected bearings of the underground survey are:

Courses.	Bearing.	Distance.
A to 1	N. 73° 03′ W.	125.53
1 to 2	N. 71° 48′ W.	166.60
2 to 3	S. 19° 12′ W.	42.00
3 to B	S. 17° 15′ W.	49.50

Yours truly,

M. F. Peltier,
Mining Engineer.

Marion, Ill., March 26, 1906.

The Wash-Borings on the Gatun Dam Site.

Sir: I have read with great interest the testimony of Prof. Burr and Mr. Stearns before the Senate Committee on Oceanic Canals, as given in your issue of March 29.

Mr. Stearns' statements are a real contribution to the knowledge of the foundations of earth dams, and his intelligent study of the subject, and his conclusions should convince the most skeptical.

Referring to that part of Prof. Burr's testimony in which he speaks of the borings taken on the site of the Gatun dam, and the frequent appearance of water flowing slowly from the outside casing of the boring pipe, though above the elevation of water in the river; and further attributing the cause of this overflow, to the material being permeable, and to water from the river entering the same: This overflow was observed, it is stated, at practically all depths, though at some borings it did not occur. The length of time the water rose in the pipe and overflowed is not stated, though this is most important in assigning a cause for the overflow.

During the past year I have had a similar experience in making wash borings through a deep deposit practically identical in character of materials with those found at Gatun, the water overflowing from the pipe, the top of which was about 2 ft. above the elevation of the water in a reservoir.

It was at first assumed that a water-bearing strata had been penetrated by the pipe, but a careful investigation showed that this was not the case, for the following reasons:

First, no permeable material was encountered.

Second, the overflow subsided in from half a minute to ten minutes.

Third, it was clearly shown that the same water was flowing from the pipe, that had previously been pumped into it to make the wash boring.

The cause of overflow was then not hard to find.

Cavities were made in the material during the boring operation, particularly at the end of the pressure pipe. When the pressure or boring pipe was withdrawn from the casing, the water began slowly to overflow from the pipe, as described by Prof. Burr, and this was undoubtedly caused by the soil slowly sloughing off and filling the cavities, expelling the water up through the pipe, its only channel of exit.

I would suggest this as the cause of the overflow at Gatun. Its correctness could be tested by simple devices.

Yours truly, Geo. B. Hazlehurst.

Catonsville, Md., March 31, 1906.

Notes and Queries.

R. H., of Cleveland, O., desires information regarding submerged sewer outlets a quarter-mile or more in length, particularly where a tidal effect is not involved. The information should include the size, length and material of the sewer, whether buried in a trench or otherwise, and special means for anchorage if any.

In our issue of March 22, p. 332, a locomotive ash-handling plant at McKee's Rocks, Pa., was referred to as the only one of its kind yet built. We have since been informed by the Link-Belt Engineering Co. that it designed and erected this type of ash-handling apparatus for the Pittsburg & Lake Erie R. R., at Groveton, Pa., and Haselton, O., in 1902 and 1903, respectively.

THE IMPROVEMENT OF STEEL RAILS.

One of the most important subjects discussed at the recent annual meeting of the American Railway Engineering and Maintenance of Way Association, at Chicago, was that of steel rails, their present defects and the necessity of improvement in their manufacture. The matter is of particular interest at this time, as the American Society of Civil Engineers has again taken up the matter, and the report of its special committee, which was presented at the annual meeting in January (Engineering News, Jan. 25, 1906), will come up for discussion at the annual convention in June. The committees of these two societies are co-operating with each other in their investigations. At the annual meeting of the American Railway Engineering and Maintenance of Way Association in 1905 an important step was taken in making the drop-test requirements more severe (Engineering News, March 30, 1905), and at the recent meeting the drop-test clause of the proposed Am. Soc. C. E. specifications was adopted. High-carbon steel was also favored, and when the representatives of rail mills suggested dangerous results as to increased breakage Mr. Webster, chairman of the committee, explained that what the specification aimed at was a high-carbon rail of better manufacture than at present, especially in the direction of the finishing being done at a lower temperature. The engineers and mill representatives also came to a difference of opinion in regard to the proposal to limit the camber to 3 ins., while in the proposed Am. Soc. C. E. specifications the limit is 5 ins. It was explained on behalf of the mills that as a matter of fact the proportion of rails having a camber of 3 ins. when they reach the hot-bed is very small, and that even with a greater camber the rails can be properly straightened and made "first-class" rails. Mr. R. Trimble, of the Pennsylvania lines, however, replied that, this being the case, it was a sufficient argument for adopting the 3-in. limit and taking all rails with a greater camber as "second-class."

No definite action was taken on the report, as the committee preferred to have such action deferred in order to see the result of the future discussion upon the proposed Am. Soc. C. E. specifications before making conclusions as to those of the American Railway Engineering and Maintenance of Way Association. The committee's recommendations as to the following points,

however, were adopted: (A) Shearing blooms 25% to prevent piping in rails; (B) adoption of the proposed Am. Soc. C. E. drop-test and shrinkage clauses; (C) adoption of the proposed Am. Soc. C. E. requirements for chemical composition of Bessemer steel rails, but with the addition of a note that "carbon may be reduced to suit local conditions"; (D) adoption of the 3-in. limit in straightening rails (instead of 5 ins., as at first suggested for the Am. Soc. C. E. specifications). We give below an abstract of the committee's report and the discussion at the Chicago meeting. As we have before pointed out, there seems to be no room for doubt that a large proportion of the modern heavy rails are distinctly unsatisfactory, and that this condition can be very materially improved by better methods of manufacture.

REPORT OF THE RAIL COMMITTEE OF THE AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.

PIPED RAILS.—We find from reports received that piped rails are frequent in occurrence. We submit the following modification of the proposed specification of the Am. Soc. C. E. on this subject:

There shall be sheared from the end of the blooms formed from the top of the ingots not less than 25%, and if, from any cause, the steel does not then appear to be solid, the shearing shall continue until it does. If, by the use of any improvements in the process of making ingots, the defect known as piping shall be prevented, the above shearing requirements may be modified.

SECTIONS.—We have no definite recommendation to make in regard to the change of the Am. Soc. C. E. section, as we have not sufficient information on this subject.

LIFE OF RAILS.—The life of rails is affected by many conditions, such as alinement, profile, density of traffic and speed of trains. We do not believe that figures prepared for one railroad will be useful to another railroad without considering the conditions mentioned above.

BREAKAGE OF RAILS.—A great deal of information on this subject has been furnished, but is largely of a confidential nature, and it is the opinion of the committee that it cannot properly be published in detail without the consent of the parties furnishing it. We can say, however, that we find that all railroads are suffering from rail failures to such an extent that we are warranted in asking the manufacturers to comply with every requirement of our specifications and to do everything in their power to improve the quality of the rail.

We give below a summary, showing the large number of failures on a few of the important railroads with a heavy traffic:

Railway No. 1.—No. of breakages of rails, 537 from Sept. 30, 1904, to Oct. 1, 1905.

Length of time rail has been in service: 1, 17 years; 4, 9 years; 43, 8 years; 22, 7 years; 3, 6 years; 3, 3 years; 22, 2 years; 439, 1 year.

Date of rolling: 1 in 1888, 1 in 1895, 31 in 1896, 17 in 1897, 24 in 1898, 4 in 1902, 21 in 1903, 438 in 1904.

General character of the breaks: Breaks between ties, 180; breaks on ties, 357. There were 224 flaws in 85-lb. rail and 80 flaws in 75-lb. rail; total flaws, 304.

Railway No. 2.—No. of breakages, 215 in the last year; 97 of these rails had seen two years' service or less; 97 were rolled in 1903; the balance from 1890 to 1905. The breaks were principally due to mashing.

Railway No. 3.—From Aug. 1, 1904, to Aug. 1, 1905, there were 196 breakages of 80-lb. steel rail. The failures arise from defective material as well as manufacture.

Railway No. 4.—No. of breakages of rails during the year 1904, 1,324 of 85 and 100-lb. rails (nothing less than 85-lbs. considered).

Length of time rail has been in service; 0 to 1 year, 182; 1 to 2 years, 95; 2 to 3 years, 105; 3 to 4 years, 67; 4 to 5 years, 79; more than 5 years, 275.

Date of rolling of those in service 5 years and less: 1896, 1; 1897, 4; 1898, 15; 1899, 80; 1900, 72; 1901, 88; 1902, 59; 1903, 189; 1904, 13.

General character of the breaks. Majority are square breaks; some crack through web (through bolt holes) and then break out through head or web. A few where cropping of ingot has been too little mash down in head with internal fissures.

Railway No. 5.—During one year 671 rails were removed which were broken or defective. A large number of these rails were removed from track before they broke. The majority of the broken and defective rails rolled in 1900 to 1904 showed piping.

Railway No. 6.—We had more than 200 rails break during the 12 months ending June 30, 1905. Our main track mileage is a little over 4,000 miles. The 1899 rails are exceptionally hard. The cause of the breakages was due to straightening at the mills, where the webs were cracked. There have been large numbers of these rails failing every year since they were laid. The breaks invariably show by longitudinal cracks in the webs. They are removed before any failure takes place as a general rule.

Railway No. 7.—In the past five years 2,855 rails failed on 400 miles of road; 20% were broken rails, 80% were mashed rails. The railway company considered the mashed rails were due to piping. Most of the failures were on 80-lb rails manufactured in 1900.

DROP TEST AND SHRINKAGE CLAUSE.—Since the last meeting of the association there has been a joint meeting of this committee with the committee on the Am. Soc. C. E. on Rail Sections, at which proposed revisions of sections and specifications were discussed. The joint committee agreed at this meeting to make no recommendations in regard to change of sections, but agreed upon the desirability of insisting upon two points in our specifications, as follows: The desirability of a drop test from each heat of steel and a shrinkage clause to control the finishing temperature. We give below a comparison of our specification with the proposed Am. Soc. C. E. specification on the drop test and shrinkage clauses, and recommend the adoption of the latter:

Am. R. E. & M. of W. Association Specification.—One drop test shall be made on a piece of rail not less than 4 ft. and not more than 6 ft. long, selected from each blow of steel. The test piece shall be taken from the top of the ingot. The rail shall be placed head upwards on the supports, and the various sections shall be subjected to the following impact tests under a free falling weight:

Weight of rail, lbs. per yard.	Height of drop.
More than 45 to and including 55.....	15 ft.
More than 55 to and including 65.....	16 ft.
More than 65 to and including 75.....	18 ft.
More than 75 to and including 85.....	20 ft.
More than 85 to and including 100.....	22 ft.

If any rail breaks, when subject to the drop test, two additional tests will be made of other rails from the same blow of steel, and if either of these latter tests fail, all the rails of the blow which they represent will be rejected, but if both of these additional test pieces meet the requirements, all the rails of the blow which they represent will be accepted.

The number of passes and speed of train shall be so regulated that on leaving the rolls at the final pass the temperature of the rail will not exceed that which requires a shrinkage allowance at the hot saws of 6 ins. for 85-lb. and 6⅛ ins. for 100-lb. rails, and no artificial means of cooling the rails shall be used between the finishing pass and the hot saws. The above shrinkage allowance may be varied, if necessary, so as to give a finishing temperature of not exceeding 1,600° F. at finishing rolls for mills rolling from rerolled blooms, and not exceeding 1,750° F. at finishing rolls for mills rolling direct from the bloom to finished rail.

Am. Soc. C. E. Specification.—One drop test shall be made on a piece of rail not less than 4 ft. and not more than 6 ft. long, selected from each blow of steel. The test piece shall be taken from the top of the ingot. The rails shall be placed head upward on the supports, and the various sections shall be subjected to the following impact tests under a free falling weight:
70 to 79-lb. rails....18 ft. 90 to 100-lb. rails....22 ft.
80 to 89-lb. rails....20 ft.

If any rail breaks when subjected to the drop test, two additional tests may be made of other rails from the same blow of steel, also taken from the top of the ingots and if either of these latter rails fail, all the rails of the blow which they represent will be rejected, but if both of these additional test pieces meet the requirements, all the rails of the blow which they represent will be accepted.

The drop-testing machine shall have a tup of 2,000 lbs. weight, the striking face of which shall have a radius of not more than 5 ins., and the test rail shall be placed head upward on solid supports 3 ft. apart. The anvil block shall weigh at least 20,000 lbs., and the supports shall be part of, or firmly secured to, the anvil. The report of the drop test shall state the atmospheric temperature at the time the test was made.

The number of passes and speed of train shall be so regulated that on leaving the rolls at the final pass, the temperatures of the rail will not exceed that which requires a shrinkage allowance at the hot saws for a 33-ft. rail of 100-lb. section, of 6 7-16 ins., and 1-16-in. less for each 5-lb. decrease of section, these allowances to be decreased at the rate of 1-90-in. for each second of time elapsed between the rail leaving the finishing rolls and being sawn. No artificial means of cooling the steel shall be used after the rails leave the rolls, nor shall they be held before sawing for the purpose of reducing their temperature.

CHEMICAL COMPOSITION.—The main differences between the Am. Soc. C. E. and the Am. R. E. & M. of W. Association specifications are as follows:

Am. Ry. E. & M. of Way:

	Per cent. 50 to 59 lbs.	60 to 69 lbs.	70 to 79 lbs.	80 to 89 lbs.	90 to 100 lbs.
Carbon	0.35-0.45	0.38-0.48	0.40-0.50	0.43-0.53	0.45-0.55
Phosphorus, not to exceed	0.10	0.10	0.10	0.10	0.10
Silicon shall not exceed	0.20	0.20	0.20	0.20	0.20
Manganese	0.70-1.00	0.70-1.00	0.75-1.05	0.80-1.10	0.80-1.10

Am. Soc. C. E.:

	Per cent. 70 to 79 lbs.	80 to 89 lbs.	90 to 100 lbs.
Carbon	0.50-0.60	0.53-0.63	0.55-0.65
Phosphorus shall not exceed	0.085	0.085	0.085
Silicon shall not exceed	0.20	0.20	0.20
Sulphur shall not exceed	0.075	0.075	0.075
Manganese	0.75-1.00	0.80-1.05	0.80-1.05

We recommend the Am. Soc.' C. E. specification, with the addition of the following footnote: "Carbon may be reduced to suit local conditions."

STRAIGHTENING RAILS.—The Am. Soc. C. E. committee has a specification in regard to straightening rails, which we think to be a desirable addition to our specification and which we recommend wth some modification.

Straightening.—Care must be taken in hot straightening the rails, and it must result in their being left in such a condition that they shall not vary throughout their entire length of 33 ft. more than 3 ins. from a straight line in any direction, when delivered to the cold straightening presses. Those which vary beyond that amount, or have short kinks, shall be classed as second quality rails and be so stamped. The distance between supports of rails in the gagging press shall not be less than 42 ins.

Rails shall be straight in line and surface when finished—the straightening being done while cold—smooth on head, sawed square at ends, variation to be not more than 1-32-in., and, prior to shipment, shall have the burr occasioned by the saw cutting removed and the ends made clean. No. 1 rails shall be free from injurious defects and flaws of all kinds.

ARRANGEMENT OF SPECIFICATIONS.—In the matter of arrangement, the Am. Soc. C. E. specification appears to be better than our form, and it is possible that after the full discussion of the former at the June convention we may find it desirable for this association to adopt the specification, with the exceptions noted above, unless we find that we can otherwise improve upon it.

OPEN-HEARTH RAILS.—There is a very strong tendency towards the use of open-hearth steel for rails, and the specification proposed by the Am. Soc. C. E. Committee, with some modifications, is offered for discussion. We are in favor of reducing the phosphorus as given in those specifications from 0.05 to 0.04. The Am. Soc. C. E. specification for open-hearth rails is as follows:

The specifications for rails made by the Basic open-hearth process shall be the same as for Bessemer rails, excepting that their chemical composition shall be

	Per cent. 70 to 79 lbs.	80 to 89 lbs.	90 to 100 lbs.
Carbon	0.53-0.63	0.58-0.68	0.65-0.75
Phosphorus shall not exceed	0.05	0.05	0.05
Silicon shall not exceed	0.20	0.20	0.20
Sulphur shall not exceed	0.06	0.06	0.06
Manganese	0.75-1.00	0.80-1.05	0.80-1.05

We recommend the following chemical composition instead of the Am. Soc. C. E. composition:

	Per cent. 70 to 79 lbs.	80 to 89 lbs.	90 to 100 lbs.
Carbon	0.63-0.73	0.68-0.78	0.75-0.85
Phosphorus shall not exceed	0.03	0.03	0.03
Silicon	0.075-0.20	.075-0.20	.075-0.20
Sulphur shall not exceed	0.06	0.06	0.06
Manganese shall not exceed	0.90	0.90	0.90

DISCUSSION.

Mr. Churchill (N. & W. Ry.)—I move the adoption of the paragraph in regard to the shearing of ingots to prevent piped rails.

Mr. Abbott (Lackawanna Steel Co.)—I do not think any association should state definitely how much should be sheared from the bloom. It will be perfectly safe to require that sufficient shearing shall be done to eliminate piping. The manufacturers are likely to oppose the 25%, and they will have very good reasons for doing so. If they can remove the piping by shearing 15% they will not agree to shear 25%. The object in shearing is to get rid of the piping, and the only matter of agreement will be to reach the point where sufficient shearing is done to remove the piping. For that reason I think the association will make a mistake to demand that 25% be shared off.

Mr. Webster. (Chairman of Rail Committee).—This matter was thoroughly discussed by the committee. They say, "If, by the use of any improvements in the process of making ingots, the defect known as piping shall be prevented, the above shearing requirements may be modified." As to going back to the old requirement, that sufficient be cut off to remove the piping, we have a large number of complaints showing that that does not accomplish the object.

Mr. Trimble (Pennsylvania Lines).—The 85-lb. rail of the Am. Soc. C. E. section is by some people called a shallow-head rail; other people have a rail which has a deeper head, and the statement has been made that the American Society rail is defective in having a shallow head. It has been stated that the head is so shallow that it fails by shearing in the track. We use the 85-lb. American Society section, and I have not seen any cases of that kind, although it has been reported to me that there have been such cases on our road. It has been stated that the 85-lb. rail with a deeper head wears better and is stronger than the American Society rail. If you will compare the Am. Soc. C. E. 100-lb. rail with other 100-lb. rails you will see there is very little difference in the depth of the head, but in the 85-lb. rail the variation in the depth of the head is very marked. We are asking now for definite information to compare the American Society's section with other sections, to find out wether we should have a deeper head or not.

Mr. Tye.—In the specifications for chemical composition I would like to see the footnote changed to read that phosphorus may be reduced where the conditions will permit. I know it is possible to get phosphorus in certain sections lower than this, but the carbon should be kept up.

Mr. Webster.—We thought we had gone about as far as we could on the phosphorus question. I know some of the roads have succeeded in getting lower phosphorus. We would like to hear the expression of the members on the phosphorus question, because it is vital.

Mr. Tye.—The Canadian Pacific Ry. has succeeded in certain sections in getting lower phosphorus than 0.085%, and I think when it is possible to get it you should get it.

Mr. Thackary.—The carbon is about right, but when you come to consider the question of low phosphorus, 0.085% in Bessemer rails, you are at once met by the practical difficulty that there does not exist in this country (or any other country from which we draw our ore supply) a sufficient quantity of ore to make the tonnage of the rail of that composition when the other needs of the trade are considered.

Mr. Webster.—The committee has given a specification which, if lived up to, would give a satisfactory rail. We have discussed it and taken the replies received in re-

gard to the breakage of rails. The present specifications are not securing us rails that are giving the service that should be expected from rails of good steel made under good conditions. We want a better rail. We have asked that the manufacturers say what the objection is to complying with the specification, and when they can comply, what percentage of their rails they can make under that requirement.

Mr. A. L. Buck.—The Canadian Pacific Ry. has in round numbers perhaps 600 miles of rail rolled practically under the Am. Soc. C. E. specifications, which are now in the second year of their service. I have examined the rails pretty closely, rails of different makers, and I believe that this chemical specification is about as near right as we need to get it. While we have failures we are having them from the same cause as in the case of the softer rails, only they are developed in a different way. We have the pipes in the rails. In our failures of rails in the past two years I have yet to find the first rail that is not broken through a pipe. I noticed in the list of roads given by the committee one road reports square breaks. I have been on expeditions of inspection and have examined a great many cases of rail breaks, and have not found a case of square break—the rail having snapped off square. Evidently that road has based its report on the section man's report, and has not investigated the matter. If these rails are examined carefully they will find a seam, a pipe, a split, wherever the break has occurred. These rails had pipes in them in the top. These top pipes failed, weakened the top of the rail, and the rail rolled out, because the metal was not hard enough to bear up under the wheel. In our new rails the metal is sufficiently hard so that the pipe does not go down. I think our troubles in regard to the top pipe will be something of the past. The pipe that is troubling is the bottom pipe. I have seen cases where this pipe has extended from 6 ins. to 18 ins. right under the web. In nearly every case it ends in a seam. The walls of the seam are smooth and the fracture will run up to within ½ in., sometimes ⅛ in., of the top of the flange, and what is left around it is good. The softness of the rail has been eliminated, and if the pipes could be eliminated I think the rail would stand as well as the old type of rail which have been held up as perfect samples.

Mr. Tye.—The phosphorus in the Canadian Pacific Ry. rails, which are made from Lake Superior ores, is the same as this specification, 0.085%; in the case of the rails made from eastern ores, the phosphorus is 0.075%. Our discard, however, was only to cut off the ragged ends, and after cutting off the point that did not seem solid to go 12 ins. further into seemingly solid material and keep on cutting until the solid material was reached. As to the shrinkage clause, I believe that even a 6-in. allowance for shrinkage is too great, because we all know that one of the most important things in getting a good rail is to have it rolled at a low temperature. The smaller your allowance for shrinkage the lower the temperature.

Mr. Griswold (Illinois Steel Co.).—I have had some experience with 80-lb. rails rolled practically to this specification. The carbon was 0.55 to 0.65% and the phosphorus 0.085%. This rail gave very poor service. It was exceedingly hard, and broke square through the section and by splitting through the web, by cracking longitudinally through the web. In a number of cases where I examined these cracks it was plain to be seen how the rails had been broken. They had been struck by the section men in gaging the track with the spike maul, and the blows from the spike maul had started a very small crack. These cracks would gradually enlarge, and finally, if the rails were not removed from the track, they would break. We made monthly inspections of the rail rolled in 1899. It was on the Louisville & Nashville Ry. We usually took the rails out before they would break. I inspected, not a great while ago, 10 rails, and nine of these had spike-maul marks. The way they came to treat the rail in that way is that the spikers had nothing with them but the gage and spike maul, and if the track is 1/16 or ¼-in. from gage they will tap it in the web with the spike maul. I believe if the rail is rolled to this specification it will prove to be a very tender rail, easily broken through shock.

Mr. Tye.—We have no such experience as this last gentleman speaks of. We have now had at least four different orders of 65,000 tons each rolled to these specifications. The Canadian Pacific Ry. runs through a rather cold country, and yet we have had no such breaks as he speaks of.

Mr. Griswold.—I think the brittleness was due to the high carbon and the phosphorus being too high for that high carbon.

Mr. Abbott.—In connection with the specification of the American Society of Civil Engineers, it would be interesting to hear from railways that have used what is regarded as high-carbon rails. The mills raise some objections to increasing the carbon in the rails under the revised specification; they can roll them, but they cannot pass the drop test called for with the same degree of safety. In other words, the engineers raise the carbon, which they anticipate will make a harder rail, and they also keep up the required drop test. Further than that, it is not altogther certain that the higher carbons thus far put out are exactly what the engineers want. I have in mind one road, a very large system, which ordered the higher carbon rails. I think they were trying for an average of 0.58%. That was in accordance with their engineer's ideas. That was granted, and these rails broke in service. The operating department immediately called up the engineer and requested him to lower his carbon, which he did.

Mr. J. H. Wallace (Southern Pacific Ry.).—We have for several years insisted on high carbon; I think our rails average between 0.55 and 0.60%, and our phosphorus is invariably down to 0.085%. We do not have any more trouble than any one else with our broken rails, and judging by the statistics given in the report of the committee, we do not have as much trouble as some other roads. A great proportion of our breakages are not due to the rail being brittle or hard, but due to some imperfection in the rail; the pipe causes the head of the rail to break down.

Mr. Webster.—In regard to high-carbon rails not meeting this drop test, we do not expect them to meet it under the present conditions of rolling. We think that all the rails are finished at too high a temperature. We want better conditions. The reason we want better conditions in rolling is that the present rails are not giving satisfaction. As a committee, we tell you what we deem would give you a better rail.

Mr. Churchill.—I move the adoption of the recommendation of the committee as to the 3-in. limit in straightening rails.

Mr. Abbott.—I am not in favor of that clause. Any mill in rolling rails will endeavor to have them come through so that by the time they are cooled they will be nearly straight. The more nearly they are straight, the better it is for the mill and the road that buys rails. They would not fill their hot bed with rails with a 5-in. camber. That would be against the interest of the mill. They would ordinarily try to keep down to 1 in. and make them perfectly straight if possible. But if by mishap rails should come down that had a camber of 3¼ ins. the mill would expect to straighten those rails and make them first quality rails, and would object to a clause of this kind in the specifications which would put such rails in the second class. The probability is that there would be a small percentage of rails that would even reach this condition; but the mills regard it as an unfair restriction to load up the specifications with matters on which the engineers and the mills cannot agree.

Mr. Trimble (Pennsylvania Lines).—Mr. Abbott has given all the argument we need to get this clause in the specification as we have made it. He says there is a small proportion of the rails which have a camber of 3 ins. when they reach the hot bed. That was the information that was given to the committee, that a large proportion of the rails did not reach anything like that figure, and we know that rails are injured in the gagging process of straightening.

Mr. Carhart.—I take it that these specifications are intended to be general and cover all sections, and a large head will of necessity demand more camber than the light head. Now it is impossible to control the shrinkage of these rails under such conditions as will allow them to come within the limits you name in this specification. Take it here in Chicago. The atmospheric conditions have a great deal to do with the matter. We have often a drop of 20°, and at times have had a drop of 40°, in a few hours, changing from a hot south wind to a north cold wind. After coming in from the hot bed, even if the rails are coming through in a perfectly normal condition, and one-half of the heat split on one hot bed and one-half on the other, the one which happens to be on the cool side, being cooled more rapidly, will of necessity show a greater amount of shrinkage than on the other. What you have named here is theoretical. That criticism would justly apply to many paragraphs in these specifications; they are too theoretical and not practical enough. As Mr. Abbott has pointed out, it is to the interest of the mills to keep the rails as straight as they can, because it lessens the subsequent work, more tonnage can be got out in a shorter space of time, and orders filled more promptly.

Mr. Webster.—Since the Am. Soc. C. E. specifications were proposed, Mr. E. F. Kenney, inspecting engineer of the Pennsylvania R. R., has made it his business to look into this matter of the straightness of rails going to the gag press of different rolling mills, and it was on his request that we reduced the 5-in. limit to 3 ins.

Mr. Abbott.—Mr. Kenney proposes that 3-in. limit in order to require the mills to adjust their camber rolls properly, and not allow a lot of rails to go down with that excessive camber. The mills would ordinarily not do that; they would prevent it in their own interest, but in a large mill there would be many rails pass through before that could be done. If it develops that the cambering roll is cambered too much it is not a practicable thing to fix the roll without stopping the mill. It will go until it is run out and it will run down a certain number of rails, perhaps 30 to 40 rails. They would then stop and correct the camber rolls, but before they could do that they would have 30 to 40 rails on the hot bed with too large a camber, according to these specifications. The mill would claim that they could put these rails in good order, make them straight, as they are required. They would feel they were restricted too much with a limit of 3 ins., and would not want to grant it with the danger of losing the tonnage of rails which I describe.

Mr. Webster.—Mr. Kenney does not want you to stop the mill, but to make quick changes. He wants good rails. If there are a lot of rails coming which he thinks are being gagged too much he wants something in the specification to say those rails should go into the seconds. He is a practical man, and he says that 3 ins. is sufficient, basing it on a record he has made from actual observation since this 5-in. clause was suggested.

REPORT ON SEWAGE PURIFICATION EXPERIMENTS AT COLUMBUS, O.

A condensed abstract of the body of the lengthy report on the sewage purification experiments at Columbus, O., during 1904-5, just issued, was published in our issue of March 29. The report was prepared by Mr. Geo. A. Johnson, Engineer-in-Charge of the Sewage Testing Station.* Mr. Johnson's summary review of the tests, with slight abbreviations, is as follows:

THE COLUMBUS SEWAGE.

The Columbus sewage from a system of combined sewers receiving certain amounts of storm water was tested with unusual thoroughness.

The analyses show that on an average the per capita composition of the Columbus sewage is much stronger than that of cities having separate sewers; and further, that it is much weaker than the sewage of those cities, such as certain cities in Massachusetts and Manchester and Leeds in England, where numerous manufacturing wastes are discharged into the public sewers. When compared with the records from other cities, the per capita constituents of the local sewage resemble most closely those of London, England. On an average the sewage, as tested, corresponds to a per capita flow of about 120 gals. daily. This dilution shows a rather stronger sewage than found in some other American cities, but it is less than half as strong as that of many European cities.

The suspended matter in the local sewage averaged as follows:

Parts per million	215
Grams per capita daily (1 gram = 0.0022-lb.)	98
Tons of dry suspended solids per 1,000,000 gals.	0.9
Tons of dry suspended solids per 1,000 population per annum	395

PREPARATORY TREATMENTS.

By preparatory treatment is meant those procedures by which there are removed the suspended matter of the sewage with as much thoroughness as economy allows on a practical scale. The coke strainers were simply rough filters or strainers of coke, while the other processes involved sedimentation in tanks of various relative sizes corresponding to periods of flow ranging from about ½ hour to 24 hours.

Briefly stated, it was found that of the suspended matter in the raw sewage, about 50 to 60 parts per 1,000,000, were so very fine that it is not feasible to remove them by any of these processes.

Screening allowed about 30 parts per 1,000,000 of coarse matters to be removed, corresponding to 0.17 cu. yds. per 1,000,000 gals.

Grit chambers [affording short periods of sedimentation.—Ed.] were not successful under the local conditions because in the sewage after screening there did not remain sufficient coarse particles of a mineral nature to allow sedimentation in small grit chambers to yield a sludge differing materially from that in the ordinary sedimentation tanks.

As to plain sedimentation it was found that about 130 parts per 1,000,000, or about 65% of the suspended matter in the screened sewage, could be deposited in sedimentation tanks holding an average flow of some four hours. Increase in the period of sedimentation beyond about four hours did not produce in a practical way much effect upon the remaining particles of suspended matter. Similarly it was found that a reduction in lineal velocity below about 4 mm. per sec. (48 ft. per hour) for a period of about four hours did not affect appreciably the removal of the suspended particles.

The treatment of the sewage by means of sedimentation to the extent above stated caused a production of sludge or solid matter upon the floor of the sedimentation tanks to the extent of about 0.65 ton of dry solids per 1,000,000 gals. This sludge contained a somewhat varying percentage of water, which averaged about 87%. This

*We are informed by Mr. Henry Maetzel, Chief Engineer of the Board of Public Service, Columbus, O., that a considerable number of the reports is available for distribution, but that the appropriation for the tests having been exhausted applications for the reports should be accompanied by the necessary postage, which is 19 cts.

makes the volume of sludge deposited by plain sedimentation tanks about 5.75 cu. yds. per 1,000,000 gals.

Chemical precipitation with the use of both lime and copperas and of sulphate of alumina was tested in sedimentation tanks having an average period of flow of eight hours. It was found that while the coagulants caused the coarser matters to be precipitated more quickly than in the case of plain sedimentation, the fine particles could be removed to a further extent only with much difficulty and expense for the precipitating chemicals. The volume of sludge is so much greater than in the case of plain sedimentation that it can be clearly stated that on the grounds of cost alone chemical precipitation is not applicable to the local sewage problem.

Plain sedimentation and the septic treatment are substantially the same in their initial step, which briefly stated consists of the deposition of about two-thirds of the total matters suspended in the Columbus sewage. Further, both effect a substantial mixing of the sewage, so that there is obtained an effluent of fairly uniform composition. The septic tanks tried at this station were of various sizes, so that the average period of flow ranged in different cases from 4 to 24 hours. In brief, it was found that the septic tanks, holding on an average about eight hours' flow, allowed this process to continue as far as it was advantageous to do on a practical scale. That is to say, septic tanks of an eight-hour capacity provide sufficient space for the accumulation of sludge, so that the putrefactive agencies characteristic of the process may liquefy and gasify as much of the solid sludge as can be disposed of in this way. It was found that on an average fully 50% of the deposited sludge in the raw (screened) Columbus sewage can be thus liquefied and gasified by the septic treatment. It was found that there would be required a sludge disposal of not more than 2.68 cu. yds. per 1,000,000 gals. with the septic treatments, as compared with 5.75 cu. yds., with plain sedimentation.

It was further found that septic tanks holding an average flow of 24 hours neither materially increased the amount of sludge septicized, nor did they cause the liquid leaving the tank to be over-septicized or of such a composition that it was with difficulty treated by subsequent filtration. No offensive odor was noticeable around the septic tanks, and the deposited septic sludge when finally removed was remarkably free from offensive odor.

One of the coke strainers, on an average, caused a removal of about 81% of the suspended matters of the applied sewage, and thus was rather more successful in regard to clarification than any of the other preparatory treatments. The other coke strainer, however, showed an average removal of suspended matter of only 64%. The coke strainers were considered to be hardly suited for adoption on a large scale at Columbus, owing to their irregular behavior and to the expense attending the removal of about 5.5 cu. yds. per 1,000,000 gals. of clogged coke which would no longer serve its purpose as a straining medium. The strainers would also be difficult to use in practice during freezing weather at rates exceeding about 1,500,000 gals. per acre daily unless they were placed under cover, which would, of course, add greatly to their expense. The removal of fats from the crude sewage by the coke strainers was about 77%; that removed by sedimentation tanks averages only about 53%.

SLUDGE DISPOSAL.

As to the question of final disposal of the sludge from preparatory treatments involving sedimentation, it is, of course, perfectly feasible to dispose of the sludge from all of the processes upon land adjoining the purification works, as is done in many of the large European cities. In regard to the sludge from the septic process it has the advantage of being not only far less in quantity than that of the other processes, but possesses several other distinct advantages in regard to its disposal. They refer particularly to the question of disposing of a portion or all of the septic sludge into the Scioto River at times of flood flows.

The advantages of the septic process are shown, first in that this is the only one of the processes in which the sludge can be satisfactorily allowed to accumulate within the purification works themselves so that it can be disposed of only during floods in the river. This means that in practice the septic tanks should be of such a size that the sludge can be readily allowed to accumulate in the tanks for eight months or more, or from the high-water stages of the early spring until rains again produce high water late in the following autumn or winter. The septic process possesses a second distinct advantage in that the storage of the sludge for such long periods as those just mentioned causes a dying out of a large number of disease germs. Hygienically this advantage is an important one and is possessed by no other process involving sedimentation.

Under conditions at many places this method of disposal of septic sludge would not be feasible, but here at Columbus the local conditions are such that each volume of septic sludge can be disposed of in the waters of the Scioto River by dilution with 800 volumes of river water. This dilution is so great that the amount of organic matter and other constituents in the river water below the purification works and coming from the Columbus sewage would then be less even than in the case of the river water at this place during low stages, when it is receiving only the settled effluent of the final filters. Another important aspect of the disposal by dilution of septic sludge is that owing to the death of a very large percentage of all of the disease germs contained in it, it is a safer and more hygienic procedure than is the discharge during storms of portions of the flows in the sewers which contain, of course, considerable numbers of virulent disease germs.

FILTRATION PROCESSES.

The several finishing processes by which the desired degree of purification may be secured of the sewage after receiving preparatory treatment comprised intermittent sand filters, which were recommended for local adoption by Mr. Hering in 1901; contact filters, which were earlier recommended by Messrs. Griggs and Alvord, and sprinkling [percolating.—Ed.] filters, which, with much success, have been recently used in Europe, but which on a large scale have not yet been used in the northern part of this country and in no case where the winter weather is as cold as it is apt to be in our Middle West and Northwest.

[Sand for] intermittent sand filters, such as is locally used for building purposes, would have to be brought from the shores of Lake Erie.

One of the most important features learned during these tests was that sprinkling filters properly designed could be operated in this northern climate during severest winter weather without any freezing of the sewage at the sprinkling nozzles.

SAND FILTERS.—After the local sewage has been screened and has had about one-quarter of the suspended matter removed from it in the grit chamber, the sand filters can purify it very thoroughly at a rate of about 100,000 gals. per acre daily. The filtered sewage would be clear and the removal of organic matter and bacteria about 90 and 98%, respectively.

With the effluent of the plain sedimentation tanks in which about two-thirds of the suspended matter of the raw sewage is removed, sand filters can operate at a rate of some 250,000 to 300,000 gals. per acre daily and produce an effluent of the character just stated.

Coke strainers in which are removed about 75% of the suspended matter in the raw (screened) sewage, allow sand filters to produce an effluent of the character stated when operated at perhaps somewhat higher rates than those last referred to.

As to the cost of operation of intermittent sand filters, the local tests were of too short duration to do other than to indicate a general harmony with experiences elsewhere on a large scale where the costs vary somewhat, but average more than 20 cts. per annum for each person connected with the sewers.

Where the effluents of septic tanks were applied to intermittent sand filters, it was found that the results approached, but did not fully equal, those obtained in conjunction with plain sedimentation. The reason was that the gas ebullition at intervals caused portions of the fine sludge to be removed from the bottom of the septic tanks and to be deposited on the surface of the sand filters. This caused more or less clogging, which would be quite a serious factor in practice unless special efforts were successfully made to keep this septic sludge from reaching the sand surfaces.

Sand filters receiving the effluents of chemical precipitation tanks holding an average flow of about eight hours suffered materially from the chemicals used in precipitating the sewage and which were not removed entirely by sedimentation. Experience elsewhere with muddy river waters of the Central West indicates that the sedimentation basins shold have a capacity of at least 24 hours. The process is less economical than others to warrant consideration locally, and the tests upon it were abandoned early in the investigations.

CONTACT FILTERS.—The effluent of the septic tanks was found to be just as capable of treatment in contact filters as in the case of unsepticized settled sewage. Every effort was made to operate these filters so that they would produce an effluent which would not putrefy and at as high a rate as this requirement would permit. With filters 5 ft. in depth it was found that with an average rate of about 1,000,000 gals. per acre daily it was hardly feasible to obtain a satisfactory effluent regularly. Double contact treatment appeared to have some advantages over single treatment. Satisfactory results as a regular procedure on a large scale could hardly be obtained at net rates materially exceeding about 600,000 to 700,000 gals. per acre daily. In this regard contact filters were distinctly inferior to sprinkling filters.

SPRINKLING [PERCOLATING] FILTERS.—With the sewage distributed from stationary nozzles over filters of broken stone 5 ft. in depth, the head of sewage on which was about 4 ft., it was found perfectly feasible to obtain a non-putrescible effluent at an average net rate of about 2,000,000 gals. per acre daily, and moreover no difficulty was experienced in operating these nozzles under severe winter conditions. The septic effluent behaved as well as that obtained from plain sedimentation tanks, but no better. Although odors were perceptible near the sprinkling nozzles, it is not thought that such odors will be noticeable at a distance of over one-quarter of a mile from the main works when built. While the effluent was non-putrescible it contained more or less turbidity, part of which was due to the extreme fine sub-microscopic particles in the original sewage, and in part was composed of fairly large particles detached at irregular intervals from the filtering material and in which they had been present as films. It was found desirable to remove by sedimentation the larger of these particles contained in sprinkling filter effluents. This was readily done in small settling basins holding an average flow of about one hour. The volume of the sediment, containing about 90% of water, would average about 2.3 cu. yds. per 1,000,000 gals.

THE PROCESS RECOMMENDED.

Without going into further detail of these several processes, it may be said that the data conclusively show the best method for adoption under local conditions to be as follows:

(1) Preliminary clarification of the sewage in basins holding on an average about an 8-hour flow and operated on the basis of the septic treatment.

(2) Purification of the septic effluent to a non-putrescible state by sprinkling filters at an average net rate of 2,000,000 gals. per acre daily.

(3) Final clarification of the effluent of the sprinkling filters in basins holding an average flow of about two hours.

This process produces a non-putrescible effluent of satisfactory appearance and from which about 90% of the bacteria in the raw sewage are removed.

REPORT BY THE INTERNATIONAL WATERWAYS COMMISSION ON THE PROTECTION OF NIAGARA FALLS.

The present authorized draft of water for power development on the American side of Niagara Falls is about 26,700 cu. ft. per sec. On the Canadian side authority has been granted for drawing about 34,200 cu. ft. per sec. The sum of these two amounts is about double the average quantity of water which passes over the American fall (27,800 cu. ft. per sec.), or referred to the total discharge over the falls, it is 27% of the average discharge and 33% of the low-water discharge over both falls. Of this aggregate amount nearly one-sixth, namely, 10,000 cu. ft. per sec., is to be charged against the Chicago Drainage Channel; the law ordering the construction of that canal specifies that figure as its flow at full development, though at present probably only two-thirds as much is flowing through. All the remainder of the figure above stated is appropriated at Niagara Falls for electric power development purposes. A small additional amount, not included in the figures, is required for lockage in the Erie and the Welland canals, and for small power developments on these canals.

The total actual diversion at the present time is less than one-third as much as the authorized diversion, being estimated by the International Waterways Commission at 17,800 cu. ft. per sec. against the authorized diversion of 60,900 cu. ft. per sec. The Commission states that even the present draft "has had an appreciable effect on the fall," and that with the full authorized draft "the loss will be important, but if the diversion be limited to this amount or reduced * * * it may not prove disastrous."

The Commission, a joint body convened by the national governments of Canada and the United States for the purpose of considering all joint waterway questions, and primarily the question of the spoliation of Niagara Falls, has recently completed its consideration of the latter problem, and its American members have made a report on the subject to the Secretary of War. This report has, in turn, been transmitted to Congress by President Roosevelt, with strong endorsement that its suggestions be enacted into law without delay. The meaning of the recommendation is that the Federal Government assume the guardianship of Niagara Falls, instead of, as hitherto, leaving the Falls to the fads and fancies of local legislatures.

The report of the American members contains the following suggestions:

If the falls are to be preserved, it must be by mutual agreement between the two countries. As a step in that direction, we recommend that legislation be enacted which shall contain the following provisions:

The Secretary of War be authorized to grant permits for the diversion of 28,500 cu. ft. per sec., and no more, from the waters naturally tributary to Niagara Falls, distributed as follows:

Niagara Falls Hydraulic Power & Mfg. Co.	9,500
Niagara Falls Power Co.	8,000
Erie Canal or its tenants (in addition to lock service)	400
Chicago Drainage Canal	10,000

All other diversion of water which is naturally tributary to Niagara Falls to be prohibited, except such as may be required for domestic use or for the service of locks in navigation canals, suitable penalties for violation of the law to be prescribed. The foregoing prohibition to remain in force two years, and then to become the permanent law of the land, if in the meantime the Canadian Government shall have enacted legislation prohibiting the diversion of water which is naturally tributary to Niagara Falls in excess of 36,000 cu. ft. per sec., not including the amounts required for domestic use or for the service of locks in navigation canals. It is assumed, however, that an understanding upon this subject would be reached by treaty. The object of such legislation would be to put a stop to the further depletion of the falls, and at the same time inflict the least possible injury upon the important interests now dependent upon this water power.

The amount to be diverted on the Canadian side has been fixed with a view to allowing to the companies on that side the amounts for which they now have works under construction, which are for the

Canadian Niagara Power Co.	9,500
Ontario Power Co.	12,000
Electrical Development Co.	11,200
Niagara Falls Park Railway Co.	1,500
Welland Canal or its tenants (in addition to lock service)	1,800

One of the effects of such legislation would be to give to Canada the advantage of diverting 7,500 cu. ft. per sec. more than is diverted in the United States. The advantage is more apparent than real, since the power generated on the Canadian side will to a large extent be transmitted to and used in the United States. In the negotiation of a treaty, however, the point should be considered.

The substance of this report was submitted to our Canadian colleagues before the passage of the joint resolution, with a view to uniting in a joint report under the general law providing for the commission. There was a substantial agreement in the statement of facts, and such differences as developed with respect to the recommendations which ought to be made did not seem insuperable, but our colleagues desired time for further consideration.

The report states that the Canadian power rights will yield the public a rental of $270,000 per year, which goes to the Queen Victoria Niagara Falls Park Commission; as the rights are equivalent to 415,000 HP., this gives a rental of $0.65 per horse-power-year. The power developments authorized on the American side yield the public no rental.

A HIGH-SPEED COLD SAW FOR CUTTING HEAVY BILLETS.

A circular metal saw with inserted teeth of high-speed steel was illustrated and described in our issue of Oct. 19, 1905. The accompanying cut represents a cold sawing machine fitted with a saw blade of this type, recently built for the American Locomotive Co. by the High Duty Saw & Tool Co., of Eddystone, Pa. It is intended for sawing 13-in. steel billets, of 0.60% carbon into convenient lengths for forging into locomotive driving axles; heavy forgings and castings can likewise be handled by the temporary removal of the special appliances fitted to the machine as shown for handling billets. The massive billet vise with its supplementary table for the firm support of the heavy axle billets is a special feature of the machine. The Tindel high-duty saw blade with which the machine is equipped is 48 ins. diameter, ⅝-in. thick and has 76 high-speed steel cutters inserted alternately on the opposite sides of the blade. The saw can cut to a depth of 10 ins. The machine is designed for a minimum feed of ⅛-in. per minute and a maximum of 1¼ ins. The peripheral speeds of the saw blade are 35 ft. and 50 ft. per minute. An automatic stop is provided so that the depth of cut can at all times be under the control of the operator. In trials of this machine at the builder's shops a cut of 1 in. per minute was made on 14-in. billets of 0.50% carbon steel with no indication of strain. The weight of the machine is about 10 tons.

The machine is geared with a speed reduction of 60 to 1 from the driving shaft to the saw arbor. This latter is a high-carbon steel forging running in a solid tapered bearing 30 ins. long, bushed with phosphor-bronze. To insure rigidity the ordinary split bearing is avoided, the arbor seat being cast solid on the carriage. The saddle carrying the spindle has the very large surface bearing of 30 × 26 ins. on the bed, and is gibbed at front and back with heavy phosphor-bronze taper shoes. The feed is by roller friction discs, transmitted through worm and worm-wheel to a heavy screw engaging in a long bronze bearing nut in the back of the saddle. The friction feed is preferred, as it is considered to give a better graduation of feed than can be accomplished through gearing.

The housing and bed plate of the machine are cast in one piece to insure rigidity. This plan also affords opportunity for affixing the massive vise and supplementary table by large bolts so that great rigidity of the holding appliance is obtained, it being of prime importance in high-speed sawing that there should be no vibration or spring of the material in cutting. Both vise and supplementary table are removable, leaving the bed plate of the machine available for steel castings, forgings, etc. To lubricate the cutters and keep them cool a stream of drilling compound is fed into the kerf by a rotary pump attached to the machine and operated by a small belt from the main driving shaft. The lubricating pipe follows the feed of the saw blade and directs the stream into the kerf at the cutting point.

TINDEL HIGH-SPEED COLD-SAW FOR BILLETS; AMERICAN LOCOMOTIVE CO.
High Duty Saw & Tool Co., Eddystone, Pa., Builders.

FEDERAL SUPERVISION OF INTERSTATE WATER pollution is proposed in a bill that has been drafted for introduction in Congress. The bill is the result of joint action by representatives of the district and national authorities at Washington, a local medical society and various other organizations. It provides for a commission under the jurisdiction of the U. S. Department of Commerce and Labor. In connection with the formulation of the bill, the following definitions of polluting substances have been made:

(a) Any liquid which contains a greater number of parts by weight of suspended matter, or of the salts of calcium, magnesium, iron, sodium, and potassium than that normally carried by the rivers in the country immediately surrounding, or any liquid possessing an acidity greater than that which would be produced by adding 20 parts of standard chemically pure hydrochloric acid to 1,000,000 parts of distilled water, except in cases where it can be shown that greater amounts of suspended matter, or of said salts, or a greater degree of acidity does not create damage to natural waters.

(b) Any liquid which contains a greater proportion of metallic arsenic than 0.1 part in 1,000,000 parts of water.

(c) Any liquid which contains a greater proportion of metals or metallic salts, other than those herein specified, than 20 parts in 1,000,000 parts of water.

(d) Any substance which becomes putrescible after incubation for one day at 37° C., the index of such putrescibility, after such period, being an insufficiency in the amount of oxygen free, or in organic combination in such incubated sample to combine in stable form with all the organic carbon and nitrogen present.

COUNTRY ROADS IN GREAT BRITAIN.

Two interesting papers on country roads were presented at a recent meeting of the Institution of Civil Engineers. One of these was by Mr. George R. Jebb, M. Inst. C. E., and was entitled "A Plea for Better Country Roads." Mr. Jebb stated, in opening, that notwithstanding the development of the railway system, roads were probably more used at present than ever in the past. He dwelt particularly upon the new problems in road construction and management brought to the front by motor cars, with their heavy wear on the roads, their mud splashing in winter and clouds of dust in summer. He stated that the heavy motor cars are causing anxiety to engineers in charge of the maintenance of bridges.

Attention was called to the need of better drainage to keep the roads free from the effects of water, and also to the need of sweeping to keep down the dust in dry weather and the mud in wet weather. The cost of sweeping (which, in the opinion of the author, would be better than scraping) would "be more than reimbursed by reduced cost in future repairs." The author stated that he had long been of the opinion that all country highways should be under the charge of county councils. The benefits from this would include, among other things, (1) better engineering superintendence; (2) possibility of making better provision for depots for materials and working plants; (3) more uniformity in methods of maintenance; (4) reduced cost of materials, and (5) more efficient and economical use of all equipment, "some of which might be worked by mechanical power." As an illustration of the present unsatisfactory condition of road maintenance in some districts, he cited one stretch of road only 1½ miles in length the responsibility for the maintenance of which is now divided between no less than six public bodies.

The other paper mentioned was on "Country Roads for Modern Traffic," by John E. Blackwall, Assoc. M. Inst. C. E. It dealt almost wholly with providing for motor vehicle traffic, including traction engines for hauling very heavy loads. Such engines, the author stated, are both cheaper and more reliable for all heavy work, provided fit roads were available. The use of traction engines is therefore bound to increase, as, of course, is also the use of the ordinary motor car. The problem of providing for both classes of traffic is thus stated by the author:

> The question to be solved is how are roads to be constructed so that traction engines will not render them impassable for all other traffic in winter, that motor cars running at high speed may not endanger the lives of all users of the highways, including their own occupants, by skidding on greasy mud, colliding with other vehicles, persons or animals, or failing to turn sharp corners, and may not envelop in dust during the summer everyone and everything within 20 yards of the road.

For traction engine traffic the author suggested that tar macadam, or an equivalent, would be desirable, but that in the open country, where there is plenty of room between fences and where land is not of high value, there might well be provided two separate roads, one for heavy and one for light fast traffic, with a footpath between. Between the footpath and the road for light fast traffic a wire fence would be erected. On the light traffic road motor cars, bicycles and light carriages would go, and on the heavy traffic road "carts, wagons, traction engines, motor lorries, led horses and all kinds of cattle." The minimum

width suggested for roads and footpath combined was 46 ft. between fences, giving 21 ft. for each roadway and 4 ft. to the footpath; but a greater width should be provided wherever possible. A main longitudinal drain was proposed beneath the footpath, with cross drains at intervals beneath the light road and with gutter inlets at the inner side of the heavy road. Both roadways would have a macadam surface, and the heavy roadway would be provided with a telford base, and would have a macadam made of "large gage material of a tough quality, worked in by the traffic where that can do so, i. e., where there is much traction engine or motor lorry traffic." Under other conditions long stretches of road would be top dressed, treated with binding material and rolled at the period of the year when the surface softens. It is estimated that the light roadway, composed of small-sized broken stone carefully laid and rolled, would not have to be recoated under from six to twelve years, and when recoating was necessary the use of tar material was advised. Prior to such recoating the author was of the opinion that, although the light roadway would not be dustless, yet, having been relieved of all heavy traffic, the dust would be much less than under other conditions.

THE REINFORCED CONCRETE BATH HOUSE FAILURE AT ATLANTIC CITY, N. J.

The collapse during construction last week of a reinforced concrete structure belonging to Reed's Bathing Establishment at Atlantic City, N. J., was briefly noted in the newspaper press dispatches. Through an engineer correspondent who examined the collapsed structure, we have secured the accompanying photographs of the failure, and the following description of the causes which led to it:

Doubtless much controversy will arise among the various concrete interests as to the reason for the failure of the reinforced concrete work being installed at Reed's Bathing Establishment at Atlantic City. When work of this nature is to be constructed in towns and districts where sufficient building regulations and a competent inspector are not provided, it is incumbent upon the architects to employ able engineers, and to give the work most careful superintendence, so as to avoid any possibility of failure which can so easily result when the erection is attended by negligence and improper inspection. In the instance of the work above cited, a most remarkable combination of conditions clearly expresses every reason and cause for the collapse.

By referring in Fig. 1 to the girder indicated by A, it is evident that the upper and lateral surfaces of the beam are badly spawled and broken. Upon inspection there was found on all surfaces of the girders in this section a frost penetration of varying depths. The concrete was so badly affected that it was possible to break off and crush large portions by a slight effort of the hand. The effect of this disintegration is more clearly evident in the section of columns shown in the foreground on the right of the picture, and perhaps most effectively illustrated by what is left of the long span girder seen in the immediate foreground.

FIG. 1. VIEW SHOWING WRECK OF REINFORCED CONCRETE BATH HOUSE, ATLANTIC CITY, N. J.

Coupled to this was a circumstance for which there can be no possible excuse, it being due to gross carelessness. In concrete construction the thickness of the floor slabs is always included in computing the effective depth of the girders, so great care must necessarily be taken that the concreting of the beams and slab be done at the same time in order that they shall be thoroughly incorporated and so give an absolute bond between these two parts. That the work was not built with this point in view is evidenced by the fact that in a number of instances the slab had entirely parted from the girder and in these cases showed little or no cohesion between the masses. This fact is further proven by the fact that the girders show a concrete of trap, while the slab is built of a limestone rock. This condition alone indicates the practical ineffectiveness of a girder for a 22-ft. span with an efficient depth of about 12 ins., not considering that the reinforcement had been placed for a girder of much greater depth.

By referring to Figs. 2 and 3, it will be seen that in two instances the girders have sheared at the point of bearing. This indicates that the reinforcement bars had not been carried to the point of bearing, thus weakening the beams and not providing the strength required at this point.

Attention should be called to the very poor character of the bearing walls which were without doubt partly responsible for the widely extending destruction. At the right of Fig. 1 may be seen a series of latticed frames

FIG. 2.

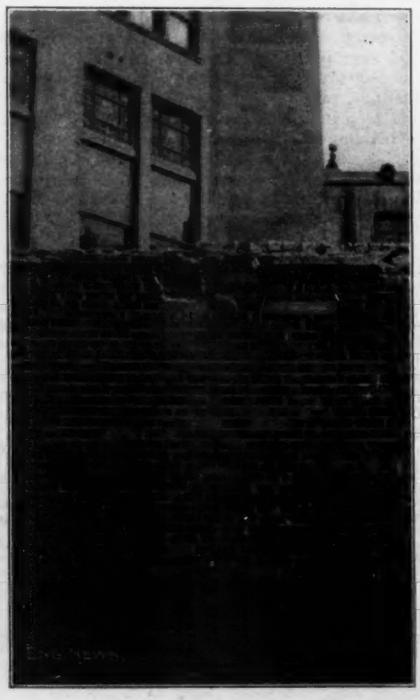


FIG. 3.

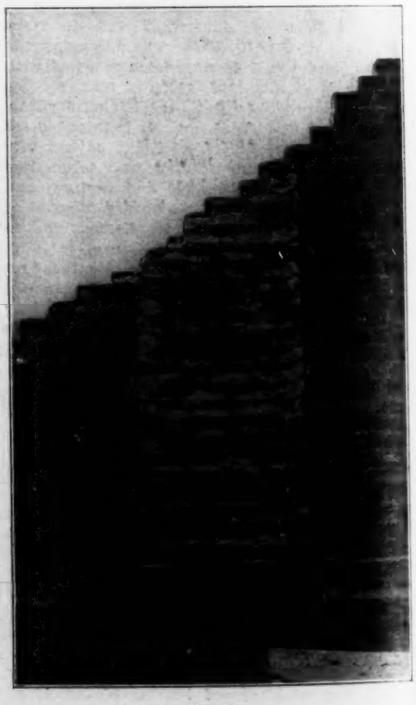


FIG. 4.

FIGS. 2 TO 4. VIEWS SHOWING DETAIL FAULTS OF CONSTRUCTION IN REINFORCED CONCRETE AND BRICKWORK.

narrow piers separated by piers and relieved only by the use of arches or lintels. Over these rests an 8-in. wall which in its turn supports the slab. The narrow, intermediate piers are further reinforced by an interior 4-in. pilaster. An examination of this wall furnishes a rather startling revelation.

By referring to Fig. 4 it will be seen that there is no bond whatever between the two outer courses of the wall, and in the instance of the pilaster the print clearly expresses that this was an after thought and was applied merely to the surface of the wall with no bond or toothing showing any intention of a bond. Thus it results that the slab on this side was wholly dependent on the strength of three 4-in. walls laid side by side in 2-ft. piers with nothing but a poor mortar to lend adhesion to the bricks. The mortar was most faulty in character, being quite lumpy, with indications of little or no cement.

It is more unfortunate that accidents of this kind should occur to damage the ever-growing popularity and desirability of concrete construction, when they result merely through the carelessness and negligence of those who have the work in hand.

NOTES FROM THE ENGINEERING SCHOOLS.

WORCESTER POLYTECHNIC INSTITUTE.—The Washburn shops have received from the Liege Exhibition the first prize for their exhibit of drill grinders. The prize was a gold medal, and was the only one given to any school shop, either trade or engineering, for a manufactured product. The shops have recently received a number of orders for machines for driving screw spikes for railway ties.

UNIVERSITY OF WASHINGTON.—One of the four timber testing stations of the U. S. Forest Service will be located here and will be under the charge of Mr. Rolf Thelen. His first work will be a comprehensive series of tests of Western hemlock, with the idea of showing that it is decidedly superior to the Eastern hemlock. The University, we may state for the benefit of Eastern readers, has a fine location on a large tract of land at Seattle, with a half-mile or so of water front.

PURDUE UNIVERSITY.—The junior students in engineering, to the number of 150, made an inspection trip to Chicago and vicinity on March 27-29, inclusive. The students were divided between the three departments as follows: Mechanical, 64; Electrical, 52; Civil, 34. Dean W. F. M. Goss, Profs. C. P. Matthews, W. D. Pence, and seven others, making ten professors in all, accompanied the students. A large number of engineering works were visited. Each student in each department was provided with a coupon schedule, giving his number, the name of the professor in charge, the rules and regulations governing the trip, the signature of the student holding the coupon ticket, and a countersignature by the professor in charge of the party. All the foregoing statements were on the stub. Separate coupons were provided for each shop or other place to be visited, giving the date and hour of departure, and the railroad, if any, over which the trip was to be made. Another coupon served as a card of introduction to a half-dozen hotels, giving the reduced rates and the various conditions which could be obtained at each, and providing for the signature of the student in the presence of the hotel clerk. The final coupon, or really the first one, since the coupons were in the reverse of the order named, was an exchange coupon for transportation from Lafayette, Ind., to Chicago.

ENGINEERING MATERIAL PRICES.

TRACK FASTENINGS—

Pittsburg f. o. b. cars—Angle bars, 1.50 to 1.75 cts.; spikes, 2.10 cts.; bolts, square nuts, 2.55 to 2.60 cts.; bolts, hexagonal nuts, 2.70 to 2.75 cts.

RAILS, STEEL—

New York—at mill—New standard sections $28; standard, second quality, $27; light sections, 12 to 25 lbs., $26 to $35; old iron, f. o. b. New York, $20 to $21; old steel, long lengths, $16.50 to $17; old steel, short pieces, $15 to $16.

Pittsburg—Standard sections, at mill, $28; standard, second quality, $27; light rails, 12 to 25 lbs., $27 to $35.

STRUCTURAL MATERIALS—

New York—at tidewater—Beams and channels, up to 15-in., 1.84½ cts.; over 15-in., 1.84½ cts.; angles, 3 × 2 and up to 6 × 6, 1.84½ cts.; zees, 3-in. and larger, 1.89½ cts.; tees, 3-in. and larger, 1.89½ cts.; steel bars, half extras, 1.74½ cts.

PLATES AND SHEETS—

New York—at tidewater—Sheared plates, tank, 1.74½ to 1.84½ cts.; flange, 1.84½ to 1.94½ cts.; marine, 2.14½ to 2.24½ cts.; still bottoms, 2.04½ cts.; fire box, 2.24½ to 3.05 cts.

Pittsburg—Tank, ¼-in. thick and heavier, 1.60 cts.; black sheets, 27 gage, 2.25 to 2.30 cts.; black sheets, 28 gage, 2.35 to 2.40 cts.

IRON—

Pig Iron:

New York—tidewater—No. 1 X Foundry, Northern, $18.25 to $18.75; No. 2 X Foundry, Northern, $17.75 to $18.25; No. 2 Plain Foundry, $17.25 to $17.75; Gray Forge, Northern, $16.25 to $16.75; Basic, Northern, $18 to $18.25; Basic, Alabama, $18.50; Basic, Virginia, $19.10; No. 1 Virginia Foundry, $18.10 to $18.60; No. 2 Virginia Foundry, $17.90 to $18.10; No. 1 Foundry, Southern, $18.25 to $18.50; No. 2 Foundry, Southern, $17.75 to $18.25; No. 3 Foundry, Southern, $17 to $17.50; No. 4 Foundry, Southern, $16.75 to $17.25; No. 1 Soft, Southern, $18.25 to $18.50; No. 2 Soft, Southern, $17.75 to $18; Gray Forge, $16.25 to $16.50; Mottled, $15.75 to $16.

Chicago—Lake Superior Charcoal, $19.75 to $20; Northern Coke Foundry, No. 1, $19.50 to $19.75; Northern Coke Foundry, No. 2, $19 to $19.25; Northern Coke Foundry, No. 3, $18.50 to $18.75; Northern Scotch, No. 1, $20 to $20.25; Ohio Strong Softeners, No. 1, $20.05 to $20.30; Ohio Strong Softeners, No. 2, $19.55 to $19.80; Southern Coke, No. 1, $18.25 to $18.40; Southern Coke, No. 2, $17.65 to $17.90; Southern Coke, No. 3, $17.15 to $17.40; Southern Coke, No. 4, $16.65 to $16.90; Southern Coke, No. 1, Soft, $18.15 to $18.40; Southern Coke, No. 2 Soft, $17.65 to $17.90; Southern Gray Forge and Mottled, $16.15 to $16.40; Malleable Bessemer, $19 to $19.25; Standard Bessemer, $19.30 to $19.55; Jackson Co. and Kentucky Silvery, 6%, $21.30; Jackson Co. and Kentucky Silvery, 8%, $23.30; Jackson Co. and Kentucky Silvery, 10%, $25.30.

Pittsburg—f. o. b. cars—No. 2 Foundry, $18.35 to $18.85; Gray Forge, $17.35; Bessemer Iron, $18.60; Ferro-Manganese, $125 to $100; Muck Bars, $32; Iron Bars, $1.95 to $2.

Cast-Iron Pipe:

New York—Carload lots, net tons, 6-in., at tidewater, $30.50.

Chicago—4-in. water pipe, $31; 6-in. and larger, $30, with $1 extra for gas pipe.

MISCELLANEOUS MATERIALS—

Asphaltum:

Ventura and other California asphalts, $19 to $21 per ton at New York; Trinidad refined, $20 to $25 per ton; Venezuela asphalt, $25 to $30 per ton; Bermuda asphalt, $25 to $30.

Cement:

Rosendale or Natural, in wood, 95 cts. per bbl.; Portland domestic, in sacks, $1.20 to $1.30; in wood, $1.50 to $1.60; German Portland, $1.90 to $2.10.

Copper:

Lake, 18.50 to 18.75 cts.; electrolytic, 18.25 to 18.50 cts.; casting, 18 to 18.25 cts.

Lead:

New York, spot, 5.35 to 5.45 cts.

Spelter:

New York, spot, 6.15 to 6.25 cts.

Tin:

New York, spot, 37.25 to 37.50 cts.

Tar:

New York, $5 to $5.50 per bbl.

Pitch:

New York, $3 to $3.20 per bbl.

LUMBER—

New York, wholesale prices:

Piling—Spruce, ordinary cargoes, 6 to 7 cts.; oak, 14-in. butt, 40 to 50 ft., 19 cts.; 50 to 55 ft., 22 cts.; 55 to 60 ft., 23 cts.; 60 ft. and up, 25 cts.; pine, 60 to 65 ft., $8.50; 65 to 70 ft., $9.50; 70 to 75 ft., $10.50; 80 ft. and up, $16.

Railroad ties—Yellow pine, 6 × 9 ins. × 8 ft., 67 to 69 cts.; 6 × 8 ins. × 8 ft., 62 to 64 cts.; 7 × 9 ins. × 8 ft., 75 to 77 cts.; 7 × 8 ins. × 8½ ft., 74 to 75 cts.; 7 × 9 ins. × 8½ ft., 79 to 80 cts.

PANAMA CANAL EXCAVATION WORK is assuming considerable proportions. There are now over 20 steam shovels at work, of which about half are excavating in the deepest part of the Culebra cut. The material is still being handled chiefly in the small Belgian cars; yet the average total cost for the material handled during February was only 68 cts. per cu. yd. Of course this represents a larger proportion of earth than will be handled as the excavation proceeds to greater depth; but on the other hand material will be handled more cheaply when large capacity cars are available. The working force is now well organized and well cared for as respects living accommodations, and the laborers have been so drilled and trained that much more satisfactory results are now being obtained than formerly. The difficulties which have been conjured up in the way of direct government work by day's labor on the canal appear to be fading away.

FLOOD DAMAGES AT THE SITE OF THE PATHFINDER dam in Wyoming were reported not long ago. Press despatches stated that the dam constructed by the U. S. Reclamation Service, near Alcova, and also a steel bridge, had been washed away and that the damage was great. We are informed that a temporary dam only was affected by the flood, that an inexpensive contractor's bridge was damaged, but not washed out, and that the total loss was probably not over $1,000. The Pathfinder Dam is being built for the U. S. Reclamation Service about ten miles above Alcova, Wyo., on the North Platte River. The contractor is the Geddes & Seerie Stone Co., of Denver. A large plant has been assembled and some stripping of quarry sites has begun. The excavation for foundations will be started after spring high water. The reservoir, when completed, will cost about $1,000,000.

A DERAILMENT CAUSED NINE DEATHS on the Chicago & Northwestern Ry., near Casper, Wyo., on March 26. An accommodation train on a new branch line ran into a washout; a steel tank car behind the caboose crushed the latter as the train suddenly stopped, and killed most of its occupants. The high water responsible for the washout also carried out the middle span of the Chicago & Northwestern bridge across the Platte River at Casper.

MUNICIPAL OWNERSHIP OF STREET RAILWAYS at Chicago was carried at the election on April 3 by a majority of 3,837, but municipal operation was defeated by a majority of 10,651. The proposition to issue $75,000,000 of certificates, under the Mueller law, for the acquisition of street railways was carried by a majority of 3,339. The constitutionality of the Mueller law will be tested in the courts.

PERSONALS.

Mr. E. L. Cruger, Asst. Engineer, Louisville & Nashville R. R., has been appointed chief clerk to Mr. W. D. Taylor, Chief Engineer, Chicago & Alton Ry.

Mr. Edward Laas, Division Supt., Chicago, Milwaukee & St. Paul, Ry., has been appointed to the newly created position of Engineer of Maintenance of Way for the entire system.

Mr. Albert Spies, M. Am. Soc. M. E., has resigned the editorship of the "Electrical Age," of New York City, his management ceasing with the publication of the February number.

Mr. L. L. Griffith, formerly Superintendent of the Bonneville Portland Cement Co., Siegfried, Pa., has been appointed Asst. Supt., Vulcanite Portland Cement Co., Vulcanite, Pa.

Mr. D. S. More, former Asst. Supt. Mexican Central Ry., has been appointed Asst. Supt. in charge of operation of the Central Ry. of Peru. His headquarters are at Lima, Peru.

Mr. A. N. Willsie has been appointed Superintendent of the St. Joseph Division, Chicago, Burlington & Quincy Railway Co., with headquarters at St. Joseph, Mo. He succeeds Mr. A. T. Perkins, resigned.

Mr. Samuel H. Lea, M. Am. Soc. C. E., has been appointed State Engineer for the state of South Dakota. Mr. Lea is the first appointee, the office having been created by the State Legislature of 1905.

Dr. James Douglas, of New York City, M. Henri Le Chatelier, of Paris, France, and Andrew Carnegie, of New York, N. Y., have been elected Honorary Members of the American Institute of Mining Engineers.

Mr. W. G. Arn, Jun. Am. Soc. C. E., has resigned as Roadmaster of the Cumberland Valley Division, Louisville & Nashville R. R., and has associated himself with the Southern Bitulithic Co., of Birmingham, Ala., as Superintendent.

Mr. R. A. Gray, who until lately was Locating Engineer for the Bolivian Government, has been appointed Construction Engineer for The Peruvian Corporation. He will have direct charge of the reconstruction of the Pacasmayo Ry.

Mr. Wm. H. Finley, who resigned some time ago to enter the contracting business as Vice-President and Manager of the Widell-Finley Co., Chicago, has been reappointed principal assistant engineer of the Chicago & Northwestern Ry.

Mr. Theodore Dwight, owing to the demands of private business, has resigned as Assistant Secretary and Assistant Treasurer of the American Institute of Mining Engineers. Mr. Henry W. B. Howard has been appointed to fill the vacancies caused by Mr. Dwight's resignation.

Prof. H. Diederichs and Mr. F. E. Junge, of Sibley College, Cornell University, have opened offices at 150 Nassau St., New York City, as consulting engineers. They will give exclusive attention to the subject of gas power. Prof. C. E. Lucke, of Columbia University, will cooperate with them.

Mr. J. A. Middleton, First Vice-President, Lehigh Valley Railroad Co., has been placed in charge of the operation and maintenance of the company's lines. The company's freight, coal and passenger traffic will be in charge of Mr. T. M. Jarvis, the newly elected Second Vice-President. Both gentlemen will have offices at 143 Liberty St., New York.

Mr. A. B. Little, of the Oklahoma Agricultural and Mechanical College, a graduate of the University of Vermont, is to be assistant professor of electrical engineering at Syracuse University the coming college year. Dr. C. H. Richardson, of Dartmouth, has been engaged as professor of mineralogy for the same institution. He will assume his duties next fall.

Mr. William B. Drake, Assistant to the Vice-President and General Manager of the St. Louis & San Francisco Ry., has been appointed Vice-President and General Superintendent of the 'Frisco lines in the Southwest, to succeed Mr. H. M. Fickinger, resigned. Mr. Le Roy Kramer, Chief Clerk in Vice-President and General Manager Gray's office, succeeds Mr. Drake.

The following changes have taken place in the Reclamation Service: Mr. George F. Harley, Asst. Engineer, has been assigned to duty with Mr. Jeremiah Ahern, at Cody, Wyo.; Mr. Edward Johnson, Jr., Engineer, has resigned from the service; Mr. William Sargent, Engineer, has been reassigned to duty with Mr. J. B. Lippincott on canal construction, Klamath Project, California and Oregon.

Mr. W. Boardman Reed, Assoc. M. Am. Soc. C. E., has resigned as engineer of maintenance of way and buildings of the New York City Railway Co. and will now take an active part in the management of the Bishop Gutta Percha Co., of New York City, of which he has been Vice-President for several years. Although Mr. Reed is to be engaged in manufacturing he will probably do considerable consulting and expert work for street railways.

Obituary.

Patrick J. Berry, a contractor of Trenton, N. J., died in that city March 25 of apoplexy.

Alonzo L. Parker, an engineer in the employ of Colonel A. D. Baird, died at his home in Brooklyn March 29, aged 62 years.

Richard W. Walsh, Chief Engineer of the Tonopah & Goldfield R. R., died recently in Utah of pneumonia. He was 38 years old.

Alfred M. Maner, Asst. Engineer, Southern Ry., died at a hotel in Johnson City, Tenn., March 18, of typhoid fever. He was about 26 years of age.

William Ryle, aged 61 years, died of pneumonia at his home in Singac, N. J., March 29. He was Superintendent of the Passaic Water Co. and had been President of the American Water-Works Association.

Samuel J. Hendy, President of the Joshua Hendy Machine Works, San Francisco, Cal., and a nephew of the founder of the company, died March 14. He was born in South Carolina in 1856, but removed to San Francisco in 1872. Upon the demise of his uncle in 1893, he succeeded to the office of President of the company.

Carl von Siemens, of the famous German family of engineers, brother of the late Sir William Siemens and of Dr. E. Werner von Siemens, died at Mentone March 21 from inflammation of the lungs. He was 77 years of age and at the time of his death was still Chairman of Siemens Brothers & Co., with whom he had been prominently connected for many years.

C. Martin Brill, president of the well-known firm of J. G. Brill Co., car builders, died at his home in Marion, Pa., March 31, of apoplexy. He was born in Cassel, Germany, in 1846, and about two years later removed with his parents to Philadelphia, Pa. In 1868 he joined his father, Mr. J. G. Brill, and organized the firm of J. G. Brill & Son. This firm was, in 1888, incorporated as the J. G. Brill Co., and Mr. Brill has since been its President.

F. N. Risteen, Mechanical Supt., Eastern Grand Division, Atchison, Topeka & Santa Fe Ry., died at Topeka, Kan., on March 26. He was born in 1856. Previous to entering the motive power department he worked as machinist and locomotive engineman. In 1893 he became Division Yard Mechanic of the Northern Pacific Ry. He was appointed Asst. Superintendent of Motive Power of the Chicago Great Western Ry. in 1901, and left a year later to accept the position of Mechanical Superintendent for A., T. & S. F. Ry.

J. Vaughan Merrick, M. Am. Soc. M. E., died at his home in Philadelphia March 28, aged 78 years. He was senior member of the firm of Merrick & Sons, manufacturers of gas and sugar machinery and marine engines from 1849-1870 and had charge of the designing of machinery and marine engines used in the government vessels during the Civil War. He was appointed a member of the board of experts of the United States Navy Department in 1867, and from 1870 to the time of his death was a trustee of the University of Pennsylvania. He was at one time a Vice-President of the American Society of Mechanical Engineers, and was a member of the American Philosophical Society and the Franklin Institute.

John Christian Kafer, Past-President of the Engineers' Club of New York City, died March 30 at Trenton, N. J. His death was unexpected, although his health had been poor for the past two or three years. He was born at Trenton, N. J., Dec. 27, 1842, where his father was a partner of John C. Stevens, at one time President of the United Railroad Companies of New Jersey. At the outbreak of the Civil War he was prosecuting his engineering studies and work and was appointed, in 1863, third assistant engineer in the Navy and ordered to the "Ticonderoga." In 1864 he was transferred to the "Mackinaw," and served in this vessel until the end of the James River campaign. He was next assigned to the frigate "Minnesota," which was attached to the Porter fleet for the first attack on Fort Fisher. Mr. Kafer was an old shipmate of Commodore Loring, and during the latter's term of office as Engineer-in-Chief of the Navy he was his principal assistant. He also served in a similar capacity under Admiral Melville. Mr. Kafer was instructor in the Naval Academy from 1868 to 1874 and from 1878 to 1882. In 1865 Mr. Kafer was offered the professorship of Mechanical Engineering at Cornell University. This honor he declined, and a short time after became associated with the Morgan Iron Works of New York City, serving as vice-president, superintendent, secretary and treasurer. Later he associated himself with the Quintard Iron Works. During the last month he had completed the organization of the consulting engineering firm of Kafer, Mattice & Warren. Mr. Kafer has been actively identified with the Engineers' Club since its organization, serving for a long time as vice-president and chairman of the House Committee, and as president from 1901 to 1904. He had also served as member of the Board of Managers and vice-president of the American Society of Mechanical Engineers. He was senior member of the Institution of Naval Architects of Great Britain, a member of the American Society of Naval Engineers, the Society of Naval Architects and Marine Engineers, and of the Engineers', Army and Navy, and Players' Clubs, New York City.

ENGINEERING SOCIETIES.

COMING MEETINGS.

IOWA ELECTRICAL ASSOCIATION.
April 18 to 20. Annual meeting at Des Moines, Ia. Secy., Geo. S. Carson, Iowa City, Ia.

AMERICAN RAILWAY ASSOCIATION.
April 25. Semi-annual meeting at Chicago, Ill. Secy., W. F. Allen, 24 Park Place, New York City.

AMERICAN ELECTROCHEMICAL SOCIETY.
May 1 to 3. Annual meeting at Ithaca, N. Y. Secy., S. S. Sadtler, 39 South 10th St., Philadelphia, Pa.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
May 1 to 4. Semi-annual meeting at Chattanooga, Tenn. Secy., F. R. Hutton, 12 West 31st St., New York City.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
June 5. Annual meeting at Atlantic City, N. J. Secy., W. C. L. Eglin, 136 Liberty St., New York, N. Y.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
June 5 to 7. Annual convention at Cleveland, O. Secy., Richard Moldenke, Watchung, N. J.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 18. Annual convention at Atlantic City, N. J. Secy., J. W. Taylor, Old Colony Bldg., Chicago, Ill.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 26. Annual convention at Thousand Islands, N. Y. Secy., Chas. Warren Hunt, 220 West 57th St., New York, N. Y.

AMERICAN WATER-WORKS ASSOCIATION.
July 10 to 14, inclusive. Annual meeting at Boston, Mass. Secy., J. M. Diven, 14 George St., Charleston, S. C.

INTERNATIONAL ASSOCIATION FOR TESTING MATERIALS.—The fourth congress of this association will be held Sept. 3 to 8, 1906, at Brussels, Belgium.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.—Prof. F. R. Hutton, Dean of the Mechanical Engineering School of Columbia University, has resigned the Secretaryship of the American Society of Mechanical Engineers. Prof. Hutton has been the Secretary of the society since March, 1883, and under his administration the society has developed from its original very modest beginning to its present commanding position with a membership of over 3,000. It has for some time been understood among Prof. Hutton's intimate friends that he intended to resign the Secretaryship on the completion of 25 years of service. The accumulating responsibilities of the Secretary's office and the considerable enlargement in immediate prospect when the society's headquarters are transferred to the United Engineering Building have caused his recent decision to resign at once, and his resignation was in fact presented to the Council at its January meeting. At the request of the Council, however, Prof. Hutton has consented to remain with the society as Honorary Secretary, in which position he will have charge of the conduct of the meetings and similar functions for which his long experience in the society and wide acquaintance among its members preeminently fits him. A regular Secretary, however, is to be elected by the Council, who will give his entire time to the society's affairs, the conduct of its business, the editing of its publications and the supervision of its headquarters. Pending the selection of the new Secretary, the present arrangements will continue in force.

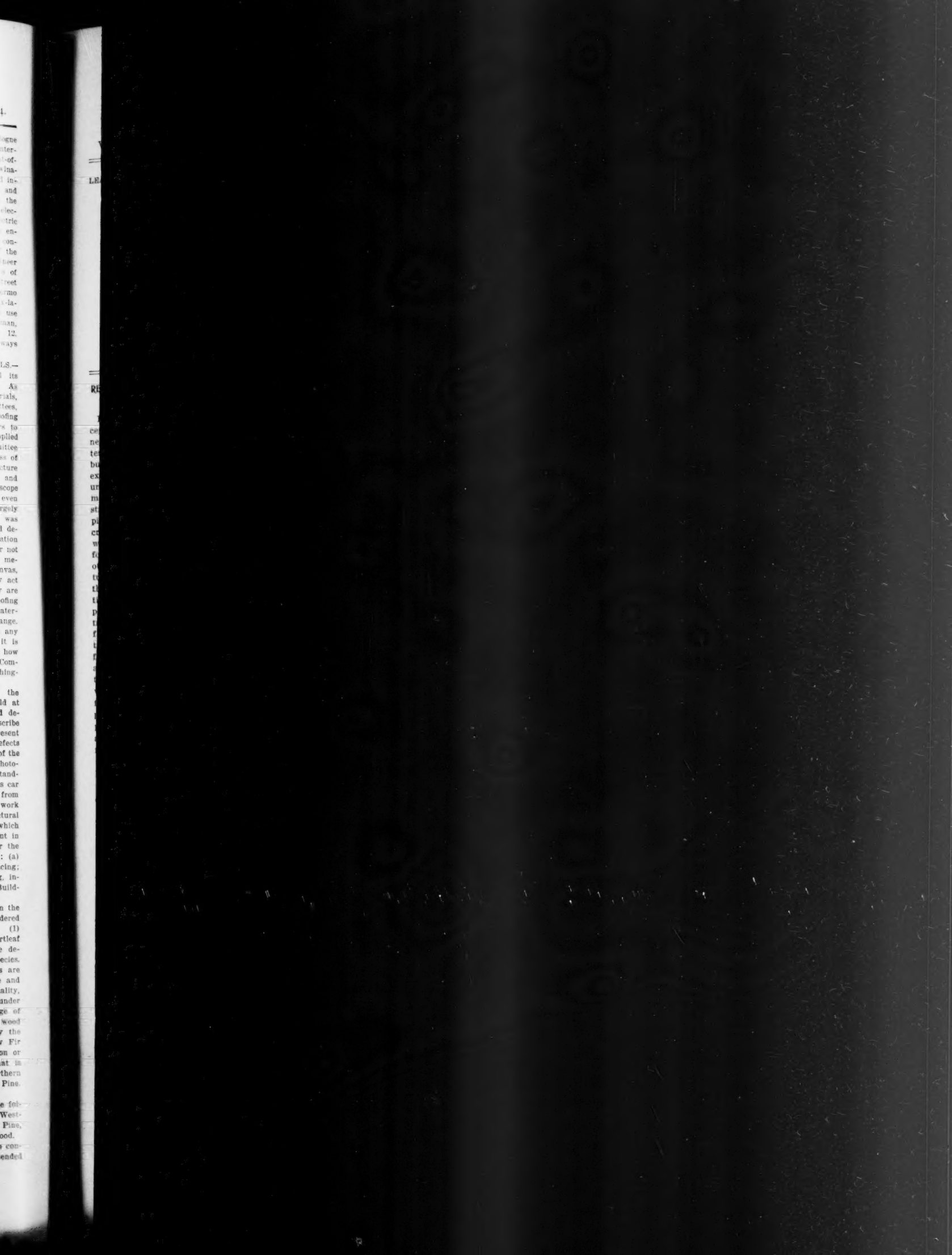
INTERNATIONAL STREET RAILWAY AND ELECTRIC RAILWAY CONGRESS.—The biennial congress of the International Tramway and Light Railway Association will be held at Milan, Italy, in September, in connection with the International Transportation Exhibition which celebrates the completion of the Simplon Tunnel. This association is to street and interurban railways what the International Railway Congress is to steam railways, but it has not as yet any extensive membership in this country. The subjects of the reports to be presented are as follows:

1. Regulation of motors for direct-current traction. 2. Mechanical brakes for electric railways; Mr. Petit, Division Engineer of the Belgian Light Railways Co., and Mr. Scholtes, Director of the Nuremburg Street Railways. 3. The dimensions of street railway cars, especially in regard to width; Mr. Geron, Director of the Cologne street railways. 4. Maximum speeds for light and interurban railways on public roads and on private right-of-way; Mr. Krasa, Inspector-General of the Buschtiehrad-Czernowitz Ry. 5. Track construction for light and interurban railways, especially in regard to rail joints and welded joints; Mr. De Burlet, Director-General of the Belgian Light Railways Co. 6. Steam turbines for electric railway power stations. 7. Progress of electric traction for street and interurban railways. 8. Gas engines for electric railway power stations. 9. Track construction of street railways; Mr. Dubs, Director of the Marseilles street railways, and Mr. Busse, Chief Engineer of the Berlin Street Ry. Co. 10. Respective merits of the zone and feeder methods of operating large street railway systems; Mr. Piazzoli, Director of the Palermo Light & Traction Co., and Prof. Rasch, of the Aix-la-Chapelle Polytechnic. 11. Results obtained from the use of current meters on electric cars; by Mr. Waltman, Director of the Cologne municipal street railways. 12. Water softening plants for branch and light railways operated by steam locomotives.

AMERICAN SOCIETY FOR TESTING MATERIALS.—The Committee on Waterproofing Materials held its second meeting in New York City on Feb. 28, 1906. As there are two distinct classes of waterproofing materials, the committee has been divided into two sub-committees, A and B. Sub-committee A is to study waterproofing materials that are mixed with cements or mortars to render them impervious and also washes that are applied to the exterior of concrete and masonry. Sub-committee B is on bituminous waterproofing, that is, that class of waterproofing where a bitumen is applied to a structure either with or without some medium such as felt and paper and specially prepared cloth. As to the scope of the work of the committee, it was decided that even though the success of the waterproofing often largely depends on the proper designing of the structure, it was hardly the province of the committee to recommend designs for waterproofing construction. The investigation should be along the following lines: (a) Whether or not the waterproofing materials act injuriously on the mediums with which they are applied, such as felt, canvas, etc., or if applied direct to the structure will they act injuriously on the concrete or metal to which they are applied; (b) a study of the permanency of waterproofing materials under surrounding conditions such as water-gas drip and aging by oxidation and by molecular change. It will be of great aid to the committee to receive any information on the subject of waterproofing, and it is asked that any one having information, no matter how little, will communicate with the Secretary of the Committee, Mr. A. W. Dow, District Building, Washington, D. C.

Committee on Standard Specifications for the Grading of Structural Timbers.—At a meeting held at St. Louis, Mo., it was decided to define standard defects occurring in structural timbers, to describe these and to present a statement as to the present knowledge concerning the influence which these defects have on the strength of the timber. The description of the defects is to be illustrated by a series of typical photographs. It was also agreed to attempt to arrange standard sizes for the various structural products, such as car sills and bridge stringers, when they are made from the same species and carry the same loads. The work of the committee will be strictly limited to structural timbers, the latter term covering those products in which the strength of the timber is the controlling element in their selection and use. It was agreed to consider the following structural products: (1) Trestle Timbers: (a) stringers, (b) caps, (c) posts, (d) mud sills, (e) bracing; (2) Bridge Ties; (3) Guard Rails; (4) Car Framing, including sills and upper framing; (5) Framing for Buildings; (6) Ship Timbers; (7) Cross Arms for Poles.

The committee decided to present for consideration the following species of wood, which are to be considered structural woods, and the following trade names: (1) Southern Yellow Pine—(a) Longleaf Pine, (b) Shortleaf Pine. It is understood that these two terms are descriptive of quality rather than of botanical species. Thus, Shortleaf Pine would cover such species as are now known as North Carolina Pine, Loblolly Pine and Shortleaf Pine. Longleaf Pine is descriptive of quality, and if Shortleaf Pine or Loblolly Pine is grown under such conditions that it produces a large percentage of hard summer wood so as to be equivalent to the wood produced by true Longleaf, it will be covered by the term Longleaf. (2) Douglas Fir—To cover Yellow Fir and Red Fir, whether grown in Washington, Oregon or any other state. (3) Norway Pine—To cover what is known as Red Pine. (4) Hemlock—To cover Northern and Western Hemlock. (5) Spruce. (6) White Pine. (7) White Oak.

The proper designation and structural use for the following species was left for further consideration. Western Yellow Pine, otherwise known as Ponderosa Pine, and Western White Pine—Tamarack, Larch, Redwood.

A series of sub-committees have been appointed to consider the standard sizes which should be recommended for the various structural elements.